0565

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

LEVEL II

DELAWARE RIVER BASIN

INDIAN FIELDS BRANCH, CUMBERLAND COUNTY

NEW JERSEY

EAST LAKE DAM

PHASE I INSPECTION REPORT
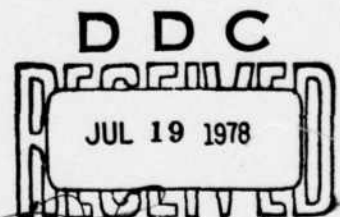# NATIONAL DAM SAFETY PROGRAM

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DEPARTMENT OF THE ARMY
PHILADELPHIA DISTRICT, CORPS OF ENGINEERS
CUSTOM HOUSE - 2D & CHESTNUT STREETS
PHILADELPHIA, PENNSYLVANIA 19106
MARCH 1978

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19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

National Dam Safety Program
Dam Inspection Report Phase I
East Lake Dam, N.J.
Dams - N.J.

ABSTRACT

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report cites results of a technical investigation as to the dam's adequacy. The inspection and evaluation of the dam is as prescribed by the National Dam Inspection Act, Public Law 92-367. The technical investigation includes visual inspection, review of available design and construction records, and preliminary structural and hydraulic and hydrologic calculations, as applicable. An assessment of the dam's general condition is included in the report.

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ABSTRACT

DEPARTMENT OF THE ARMY
PHILADELPHIA DISTRICT, CORPS OF ENGINEERS
CUSTOM HOUSE—2 D & CHESTNUT STREETS
PHILADELPHIA, PENNSYLVANIA 19106

IN REPLY REFER TO

NAPEN-D

16 JUN 1978

Honorable Brendan T. Byrne
Governor of New Jersey
Trenton, NJ   08621

Dear Governor Byrne:

Inclosed is the Phase I Inspection Report for East Lake Dam in Cumberland
County, New Jersey which has been prepared under authorization of the
Dam Inspection Act, Public Law 92-367. A brief assessment of the dam's
condition is given on pages 1 and 2 of the report.

The inspection indicates the dam's earth embankment to be in poor condition
and the dam's spillways and outlet channel to be seriously inadequate. To
insure adequacy of the facility, the following actions, as a minimum, are
recommended:

    a. Hydraulic and hydrological investigations should be initiated, with-
in three months after the date of approval of this report, to determine re-
quired corrective action(s) necessary to substantially improve hydraulic
capacities of the combined spillway facilities and outlet channel. Con-
struction of improved spillway and outlet facilities should be completed in
calendar year 1979.

    b. Within four months from the date of approval of this report, engi-
neering studies should be initiated to determine a positive means of con-
trolling seepage through the dam, adequate upstream face slope protection
and the method of installing an outlet pipe at the low point of the reser-
voir. Implementation of the results of these studies should be initiated
in calendar year 1979.

    c. Within one year from the date of approval of this report, the road
surface drainage problem on top of the dam should be rectified, the wall
on the upstream side of the roadway repaired and the settled areas atop
the dam properly filled.

NAPEN-D
Honorable Brendan T. Byrne

Two copies of the report are being furnished to Mr. Dirk C. Hofman,
New Jersey Department of Environmental Protection, the designated State
Office contact for this program. Within five days of the date of this
letter, a copy will also be sent to Congressman Edwin B. Forsythe of the
Second District. Under the provisions of the Freedom of Information Act,
the inspection report will be subject to release by this office, upon
request, thirty days after the date of this letter.

An important aspect of the Dam Safety Program will be the implementation
of the recommendations made as a result of the inspection. We accordingly
request that we be advised of proposed actions taken by the State to im-
plement our recommendations.

Sincerely yours,

HARRY V. DUTCHYSHYN
Colonel, Corps of Engineers
District Engineer

1 Incl
As stated

Cy Furn: w/incl (dupe)
Mr. Dirk C. Hofman, P.E.
Department of Envirnmental Protection

2

COHANSEY RIVER BASIN

Name of Dam: East Lake Dam
County and State: Cumberland County, State of New Jersey
Inventory Number: 00070

PHASE I INSPECTION REPORT

NATIONAL DAM SAFETY PROGRAM

Prepared by: O'Brien & Gere Engineers, Inc.
              Justin & Courtney Division

For: United States Army Corps of Engineers
      Philadelphia District

Date: March 20, 1978

PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam   East Lake Dam

          State Located New Jersey
          County Located Cumberland County
          Stream   Mill Creek
          Date of Inspection   February 3, 1978

ASSESSMENT OF
GENERAL CONDITIONS

The East Lake Dam is an earth fill structure consisting of poorly
compacted random fill and organic material. Settlement is visible in
the sidewalk on top of the dam. The upstream slope of the dam is not
properly protected against erosion and wave action. Seepage at the
downstream slope has been observed in the past by the owners of the
fabric plant located immediately downstream of the dam. To correct
this problem, a foundation subdrain system and sump pit were
installed. The surface drainage system for the road on top of the dam
is inadequate. The combined spillway facilities are unable to pass
one-half of the Probable Maximum Flood (PMF) without overtopping
the dam. The dam is not equipped with a means of draining the
bottom 8 feet of the reservoir below the principal spillway crest.

Because of the deficiencies listed above, a further investigation, as
outlined in the National Program of Inspection of Dams, Volume I,
Appendix D, Chapter 4, is recommended. The above deficiencies are
believed to be serious and the further investigation should begin as
soon as possible.

The dam has also been investigated previously by another private consultant. One of the results of that investigation was that the State of New Jersey Department of Environmental Protection directed the dam operators to lower the normal reservoir elevation by 4 feet by keeping the principal spillway gates open. Other improvements are apparently being considered for this dam but the details of the investigation or the status of the proposed improvements were unavailable due to pending litigation.

Leroy H. Heffner

The inspection indicates the dam's earth embankment to be in poor condition and the dam's spillways and outlet channel to be seriously inadequate. To insure adequacy of the facility, the following actions, as a minimum, are recommended:

a. Hydraulic and hydrological investigations should be initiated, within three months after the date of approval of this report, to determine required corrective action(s) necessary to substantially improve hydraulic capacities of the combined spillway facilities and outlet channel. Construction of improved spillway and outlet facilities should be completed in calendar year 1979.

b. Within four months from the date of approval of this report, engineering studies should be initiated to determine a positive means of controlling seepage through the dam, adequate upstream face slope protection and the method of installing an outlet pipe at the low point of the reservoir. Implementation of the results of these studies should be initiated in calendar year 1979.

c. Within one year from the date of approval of this report, the road surface drainage problem on top of the dam should be rectified, the wall on the upstream side of the roadway repaired and the settled areas atop the dam properly filled.

APPROVED: *Harry V. Dutchyshyn*

HARRY V. DUTCHYSHYN
Colonel, Corps of Engineers
District Engineer

DATE: *16 June 1978*

UPSTREAM FACE OF EMBANKMENT LOOKING WEST

TOP OF DAM LOOKING WEST—UPSTREAM SIDE

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Recommended Guidelines for Safety Inspection of Dams, Chapter 4

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
NAME OF DAM EAST LAKE DAM ID# 00070

SECTION 1 - PROJECT INFORMATION

1.1   GENERAL

a.   Authority.   This report is authorized by the Dam
Inspection Act, Public Law 92-367, and has been prepared in
accordance with contract #DACW61-78-C-0052 between O'Brien &
Gere Engineers, Inc., Justin & Courtney Division, and the United
States Army Corps of Engineers, Philadelphia District.

b.   Purpose of Inspection. The purpose of this inspection is
to evaluate the structural and hydraulic condition of the East Lake
Dam and appurtenant structures, and to determine if the dam
constitutes a hazard to human life or property.

1.2   DESCRIPTION OF PROJECT

a.   Description of Dam and Appurtenances. East Lake Dam
is an earth embankment with a maximum height of about 17 feet and
a length of about 500 feet. East Commerce Street, an asphalt paved,
two lane road, is located on the top of the embankment. The top of
the embankment varies from elevation 26.5 feet Mean Sea Level
(MSL) at the east end, to elevation 24.0 feet MSL at the west end. A
masonry wall has been constructed along the upstream side of the
road to elevation 27.5. The elevation of the base of this masonry wall
could not be determined by visual inspection or from available data.
The Murbeck Knitted Fabrics plant is located immediately down-
stream of the embankment. This plant uses water from the reservoir
for fabric processing.

The principal spillway is located on the east side of the
embankment, and consists of two gated 48 inch circular openings on
the upstream face of a concrete box structure. Rectangular overflow
openings are located on the two sides of the box structure about 4
feet above the invert of the circular openings. Outflow from the
chamber is through two 48 inch cast iron pipes through the
embankment. The downstream invert of the pipes is 5 feet above a
concrete basin used to direct flow into a box culvert.

An 11 foot wide emergency spillway is located along the west
bank of the reservoir about 150 feet upstream of the embankment.
Several weirs and culverts are located along the spillway channel
downstream.

1

An intake structure and pump are located in the reservoir near the west end of the embankment to supply water to a constant head tank for use by the Murbeck plant. Excess water flows through a return line back into the reservoir.

b. Location. The East Lake Dam is located in the city of Bridgeton, Cumberland County, New Jersey. It is located along East Commerce Street, just downstream of the confluence of Indian Fields Branch and Jackson Run. Flow from the spillway channel discharges into Mill Creek about 3,000 feet above the Cohansey River. Mill Creek is subject to tidal variations. See the vicinity and Drainage Basin Maps in the appendix (Figures 1 and 2).

c. Size Classification. The maximum height of the dam (to the top of the wall) is about 17 feet and the maximum storage capacity of the reservoir is about 230 acre-feet. Therefore, the dam is in the small size category as defined by the Recommended Guidelines for Safety Inspection of Dams.

d. Hazard Classification. A large fabric plant is located immediately downstream of the dam, with business and residential areas of Bridgeton within a mile downstream of the structure. Therefore, the dam is in the high hazard category as defined by the Recommended Guidelines for Safety Inspection of Dams.

e. Ownership. The ownership of the dam is in question. The dam may have been originally a privately owned structure. The City of Bridgeton assumed the maintenance responsibility for the road on top of the dam. However, neither the City of Bridgeton nor Murbeck Fabrics have accepted the responsibility for the dam itself.

f. Purpose of Dam. The dam is used to store water for use in fabric processing at the Murbeck Fabric plant downstream of the dam.

g. Design and Construction History. No information is available on the dam design or the construction methods used. The dam is reported to have been built in 1805.

Recent borings show that the dam was constructed of various types of earth and organic materials. It is doubtful that proper placement and compaction techniques were used to construct the dam.

h. Normal Operational Procedures. See Section 4 for information on operation procedures.

1.3 PERTINENT DATA

a.   Drainage Areas. The East Lake Dam is located just below the confluence of Jackson Run and Indian Fields Branch. These streams drain areas of 1.4 square miles and 5.9 square miles respectively. The total drainage area above the dam is 7.3 square miles.

b.   Discharge at Damsite. No discharge records are available at this site. According to the plant engineer at the Murbeck Fabric plant, both the principal and the emergency spillways were inadequate to prevent property damage adjacent to the dam during tropical storm Doria of 1972. East Commerce Street was flooded near its junction with East Avenue during this flood. The worst condition occured at high tide, when the box culvert below the principal spillway flowed under pressure, causing backup of flow through the culvert manholes in the fabric plant buildings downstream of the dam. Water in the plant reached a depth of about 10 inches above the floor during this flood.

The capacity of the principal spillway is 230 cfs when the reservoir is at the top of the wall on top of the dam. The capacity of the emergency spillway is limited by weirs, bridges, and culverts under East Commerce Street. The maximum flow in the emergency spillway, before general flooding of the surrounding area occurs, is estimated to be 20 cfs.

c.   Elevations above MSL. (from Figure 4)
Top of stone wall - 27.5 feet
Top of dam - varies between elevation 24.0 and 26.5
Principal spillway crest (invert elevation of gated openings) - 20.5 feet
Design Surcharge - 29.9 feet ($\frac{1}{2}$ PMF)
NOTE: The design flood overtops the wall by 2.4 feet

d.   Reservoir
Length of normal pool - 1800 feet (elevation 20.5)
Length of maximum pool - 2600 feet (elevation 27.5)

e.   Storage (from Figure 6)
Normal pool - 75 acre-feet (elevation 20.5)
Maximum pool - 230 acre-feet (elevation 27.5)

f.   Reservoir Surface Area (from Figure 6)
Normal pool - 17 acres (elevation 20.5)
Maximum pool - 26.5 acres (elevation 27.5)

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g. Dam
    Type - earth fill
    Length - 500 feet
    Top width - 60 feet
    Side slopes - 2 horizontal : 1 vertical (estimated)
    Zoning - unknown
    Impervious core - unknown
    Cutoff - unknown
    Grout curtain - unknown

h. Diversion and Regulating Tunnel
    None

i. Spillways (Elevation taken from drawings supplied)
    1) Principal spillway
    Two 48 inch cast iron pipes
    Length of weir - 8 feet
    Crest elevation - pipes, 20.5 feet MSL -
     weir, 24.5 feet MSL
    Gates - two 48 inch circular gates
    U/S Channel - none
    D/S Channel - concrete box culvert; 5.8 foot wide,
    5.66 foot deep

    2) Emergency spillway
    Type - earth channel with stone and masonry walls
    Length of weir - 11 feet
    Crest elevation - 24.3 feet MSL
    Gates - none
    D/S Channel - concrete culvert under East Commerce
    Street discharges into a sluiceway under the Murbeck
    Fabric plant.

    j. Regulating Outlets.A pump is used to supply water
through a pipe in the embankment to a constant head tank
downstream of the dam for use in the fabric plant. Excess water
from the tank is returned to the reservoir through an overflow pipe.

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SECTION 2 - ENGINEERING DATA

2.1   DESIGN - The information available for review of the East
Lake Dam included:

a.   Soil boring logs from tests taken in November of 1974.
A copy of the logs is included in the appendix (see Figures 7
through 14).

b.   Seven drawings from a "Study to Improve the Capacity
and Safety of East Lake and East Commerce Street Dam," by
A.G. Lichtenstein and Associates, Consulting Engineers, Teaneck,
New Jersey.   The study was jointly financed by Murbeck Knitted
Fabrics, Cumberland County, and the City of Bridgeton.   Pertinent
drawings are included in the appendix.   The study was not available
for inspection, due to pending litigation concerning ownership of the
dam.

2.2   CONSTRUCTION - No information on construction materials
or techniques is available.

2.3   OPERATION - See Section 4.

2.4   EVALUATION - The soil borings show that the dam consists of
sand, gravel and silt.   The number of blows on the 1-3/8 inch diameter
core sampler, with a 140 pound hammer dropping 30 inches, varied
from 1 to about 20 per 6 inches of penetration.   Organic material,
ashes, cinders, wood chips, sawdust and peat were all encountered in
the dam or foundation.   The groundwater was at about elevation 11.0
in most of the borings after 25 days, although it was as high as
elevation 14.2 in boring B9.   The ground downstream of the dam
varies from elevation 13 to elevation 17.7 and groundwater was
between 2.1 and 3.5 feet from the ground surface in the area.
Without the subdrain system in the foundation downstream of the
dam, this level would undoubtedly be higher.   The borings indicate
that the dam is composed of loose fill of medium to high permeability
as well as compressible material.   The drawings by A.G. Lichtenstein
show existing topography and some proposed improvements.   They
show little detail with regard to the composition or condition of the
existing facilities.   Evaluation of these plans is beyond the scope of
this investigation.

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SECTION 3 - VISUAL INSPECTION

3.1   FINDINGS

a.   Dam.   The dam is an earth embankment with a stone wall on the upstream side of the crest.   The stone wall is noticably settled and tilted.   The upstream slope consists of a loose sand fill which shows evidence of erosion and settlement.   Vertical depressions of 2 to 3 feet are evident near the water surface.   There are no training walls at the outlet structure to protect against the effects of erosion in this area.   Erosion to depths of 3 to 4 feet is apparent for a length of 30 feet west and 50 feet east of the outlet structure.   Sand has been placed in these areas to protect the embankment.

The profile of East Commerce Street on the top of the embankment is such that ponding occurs near the west end of the embankment during periods of rainfall excess.   The roadway has no effective surface drainage system in the area of the dam, so the ponded water percolates into the embankment.   There are two areas of excessive settlement along the sidewalk on the downstream side of East Commerce Street.   These areas are about 30 and 40 feet long, 3 feet wide, with settlement from 1.5 to 2.5 feet.   A similar settlement area about 30 feet by 3 feet by 9 inches deep is located near the middle of the dam on the upstream side of the road.   It was reported by the plant engineer that the water entering the settlement areas appears along the downstream face and toe as seepage.   In 1955, the plant installed French drains at the toe of the dam to remove the seepage water and to lower the water table.   A pump used to remove water from the drains, keeps the water table about 6 inches below the floor of the plant.   The extent of the drainage system is not known.   No seepage was observed coming from the downstream slope of this embankment during the inspection.

b.   Appurtenant Structures.   The principal spillway consists of two 48 inch gated openings leading to a concrete box structure.   Overflow openings are located on the sides of the box structure, 4 feet above the gated openings.   Outflow from the box structure is through two 48 inch cast iron pipes lined at the entrances with 42 inch corrugated metal pipes.   (Refer to the sketch on page A28.)   The gates, which are kept open, are operated from the top of the box structure.   The concrete in this structure is deteriorated.   The gates are badly rusted but, according to Murbeck's Plant Engineer, are operational.   Concrete has spalled in a number of places, the columns are cracked, and there is a large crack near the junction of the beam with the slab.   Repair work has been done to the downstream side of the box structure.

The emergency spillway is an earth channel about 11 feet wide constructed through the grounds of a private estate. Flow travels 400 feet before entering 18 inch and 24 inch diameter pipes under East Commerce Street. Channel obstructions include 3 bridges and 2 weirs. The ability of the emergency spillway to discharge flood waters is limited by the small culverts and obstructions along the channel.

Water is pumped to a constant head tank located in the downstream slope of the embankment through a pipe in the embankment. The condition of the pipe through the embankment was not determined during the inspection due to inaccessability.

c. Reservoir Area. The emergency spillway channel is constructed through a private estate. This area was flooded in 1972. The conditions in the reservoir area do not appear to affect the safety of the dam.

d. Downstream Channel. The downstream channels are constricted, and constructed under the plant buildings. Mill Creek is a tidal stream, which can have a large affect upon the discharge capacity of the channels.

3.2 EVALUATION Visual inspection of the site shows that the dam and appurtenant structures were not constructed according to modern standards, nor are they being maintained properly. The upstream face of the dam is not suitably protected and maintained. Areas of settlement on the dam may be due to the lack of proper surface drainage or internal erosion (piping). The pipes through the embankment were not inspected throughout their length. However, the readily visible portions appeared to be in good condition. The masonry wall on the downstream side of the outlet works appeared to be in good condition. A further discussion and evaluation of these subjects is included in Section 7.

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SECTION 4 - OPERATIONAL PROCEDURES

4.1   <u>PROCEDURES</u>   - According to the information obtained during the field inspection from the Murbeck Plant Manager and the Plant Engineer, the operational procedures in connection with the dam consist primarily of pumping water from the reservoir into the constant head tank downstream of the dam and the operation of the foundation subdrain system downstream of the dam. Maintenance of the dam has been minimal and appears to have consisted only of sandbagging or dumping sand on the upstream face of the dam to repair eroded areas.

A foundation subdrain system and sump pit have been installed downstream of the dam. Water from the sump pit is pumped into a drainage ditch downstream of the dam.

The dam is not equipped with any facilities for drawing down the reservoir below elevation 20.5 A survey of the reservoir bottom made in 1976 shows that the bottom of the reservoir is at elevation 10.0. The elevation of the ground downstream of the dam is 13.0.

In 1973, after an investigation of the dam was made by a private consultant, the State of New Jersey Department of Environmental Protection (NJDEP) directed the lowering of the normal reservoir elevation by 4 feet. This was accomplished by opening the 48 inch gates in the principal spillway. These gates must be kept open to maintain the reduced reservoir elevation required by NJDEP.

SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1   EVALUATION OF FEATURES

a.   Design Data.   The Probable Maximum Flood (PMF) hydrograph was calculated from the Probable Maximum Precipitation using standard reduction factors.   The Soil Conservation Service curvilinear unit hydrograph was developed for the drainage basin and used as a basis for constructing the PMF hydrograph. Peak inflow discharges to the reservoir for the PMF and one half of the PMF are 25,000 cfs and 12,500 cfs respectively.   These inflow hydrographs were routed through the reservoir by the modified-Puls method, utilizing computer program HEC-1. The peak outflow discharges for the PMF and one half of the PMF are 25,000 cfs and 12,450 cfs respectively.   Both the PMF and one half of the PMF result in overtopping of the dam.

The stage-outflow relation for the principal spillway was based upon the available information on invert elevations and sizes for the gated openings and the outlet conduits.   Further assumptions on discharge through this outlet are noted on page A28. The emergency spillway discharge is limited by the small pipes used to convey flow under Commerce Street. The capacity of the pipes is estimated at 20 cfs under 2 feet of head.   At this elevation, a 70 foot section of the wall along East Commerce Street is inundated.

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SECTION 6 - STRUCTURAL STABILITY

6.1   EVALUATION OF STRUCTURAL STABILITY - East Lake Dam
is constructed on unconsolidated sediment of the Bridgeton formation
(quaternary) and possibly the Cohansey formation (tertiary). These
geological units are composed of fine to coarse grained sand and
gravel and contain some thin zones of finer grained sediments (silt
and clay). The differences in the two formations are subtle and no
distinction can be accurately made at the dam site or from boring
data.

The dam is in seismic risk zone 1 of the Seismic Zone Map of
the United States. Due to the low height of the dam, the risk of
seismic damage is probably low. However, due to the character of
the embankment materials as evidenced by the borings, the possibility
of liquefaction during an earthquake may exist.

The physical condition of the dam is poor. Considerable
settlement, erosion and possibly piping of the embankment has taken
place. The stability of the upstream slope is suspect due to the
existence of a loose sand fill and the tilted wall. The risk of internal
erosion of the embankment and the foundation is high. The dam and
abutments are also highly susceptible to surface erosion although the
paved roadway will offer some resistance.

Shrub and tree growth on the downstream slope of the
embankment makes it difficult to tell if sloughing of the slope is
taking place. No leakage was observed flowing out of the
embankment slopes; however, this may be because the drainage is
controlled by the foundation subdrain system downstream of the dam.
The fact that the top of the dam is over 60 feet wide also has a great
influence in controlling the seepage and reduces the danger of slope
or piping problems. However, these possibilities still exist.

The structural condition of the principal spillway is also
questionable. The concrete in the outlet structure showed evidence
of substantial cracking and deterioration.

SECTION 7 - ASSESSMENT/REMEDIAL MEASURES

7.1   DAM ASSESSMENT - The East Lake Dam is deficient in the following respects:

a.   The embankment contains poorly compacted, pervious soil and organic material. These materials are unable to control the embankment seepage and settlement properly.

b.   The embankment has inadequate slope protection on the upstream face.

c.   The surface drainage system for the road on top of the dam is inadequate.

d.   The dam is not equipped with facilities to empty the reservoir in an emergency.

All of the above deficiencies are potentially dangerous. However, the dam has withstood the test of time without serious consequences. This may be due to the low head to which the embankment is subjected, the unusually wide top width of the dam, the subdrain system downstream of the dam and other factors which cannot be evaluated within the scope of a Phase I Investigation.

In addition to the deficiencies in the embankment listed above, the spillways are inadequate. For a flood of one half of the PMF, the wall on top of the dam is overtopped by 2.4 feet.

For the above reasons, an additional investigation as outlined in the National Program of Inspection of Dams, Volume I, Appendix D, Chapter 4, is recommended to be made of the East Lake Dam as soon as possible.

7.2   REMEDIAL MEASURES - The following remedial measures could be considered for improving the safety of the dam:

a.   Construction of a positive means for controlling seepage through the dam such as:

1)   An impervious earth core with properly designed filters constructed by conventional or slurry trench methods.

2)   An impervious liner on the upstream face.

3)   Steel sheet piling.

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b. Addition of slope protection for the upstream face (riprap with a filter blanket or equivalent).

c. Filling of settled areas with compacted earth material and improvement of the surface drainage system.

d. Repair of wall on the upstream side of the roadway.

e. Construction of a new spillway.

f. Installation of a pipe at the low point of the reservoir for drainage purposes.

FIGURES

FIGURE 1
REGIONAL
VICINITY MAP
SCALE 1:24,000
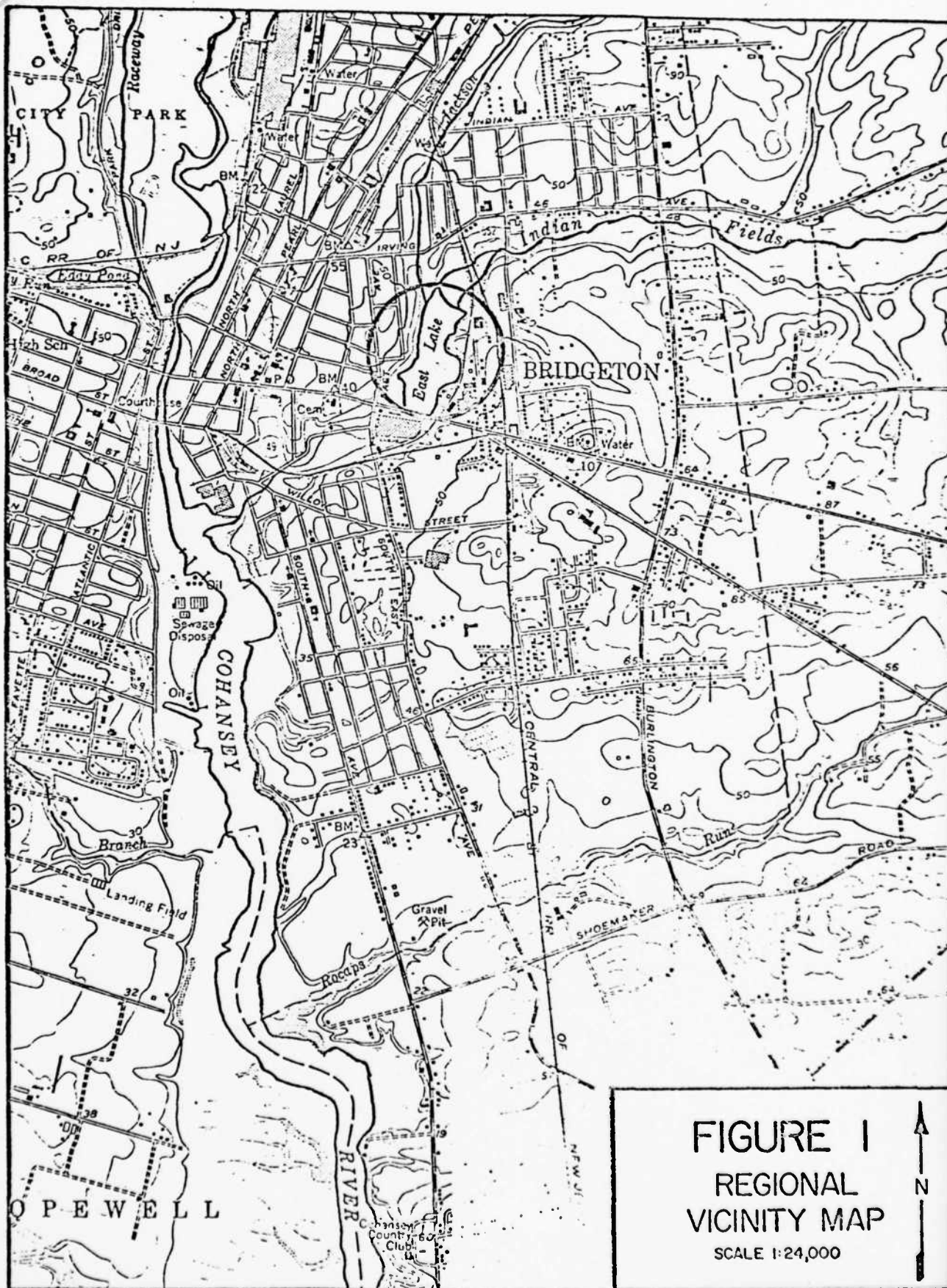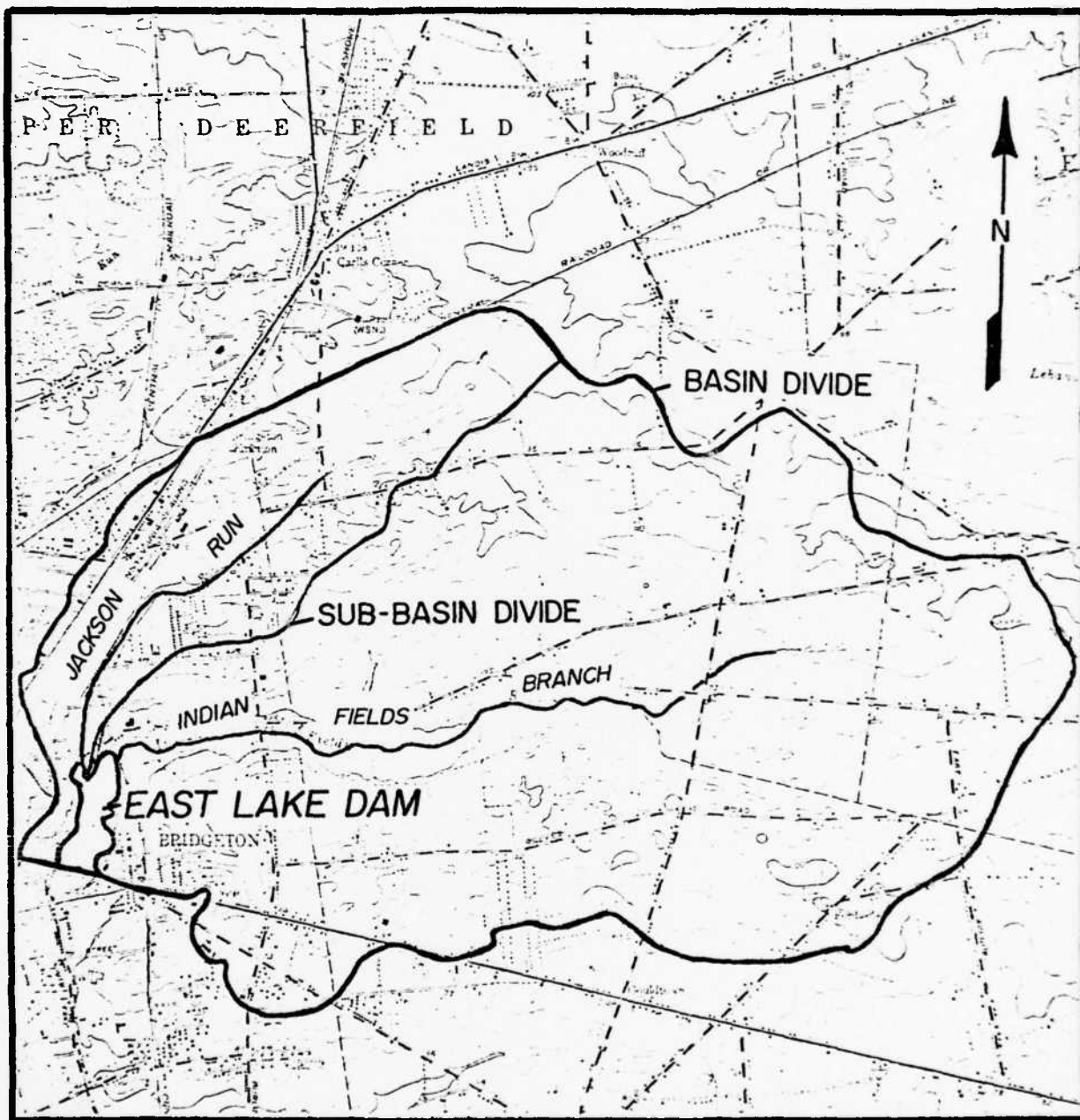
SCALE 1" = 4,000'

FIGURE 2

DRAINAGE BASIN MAP
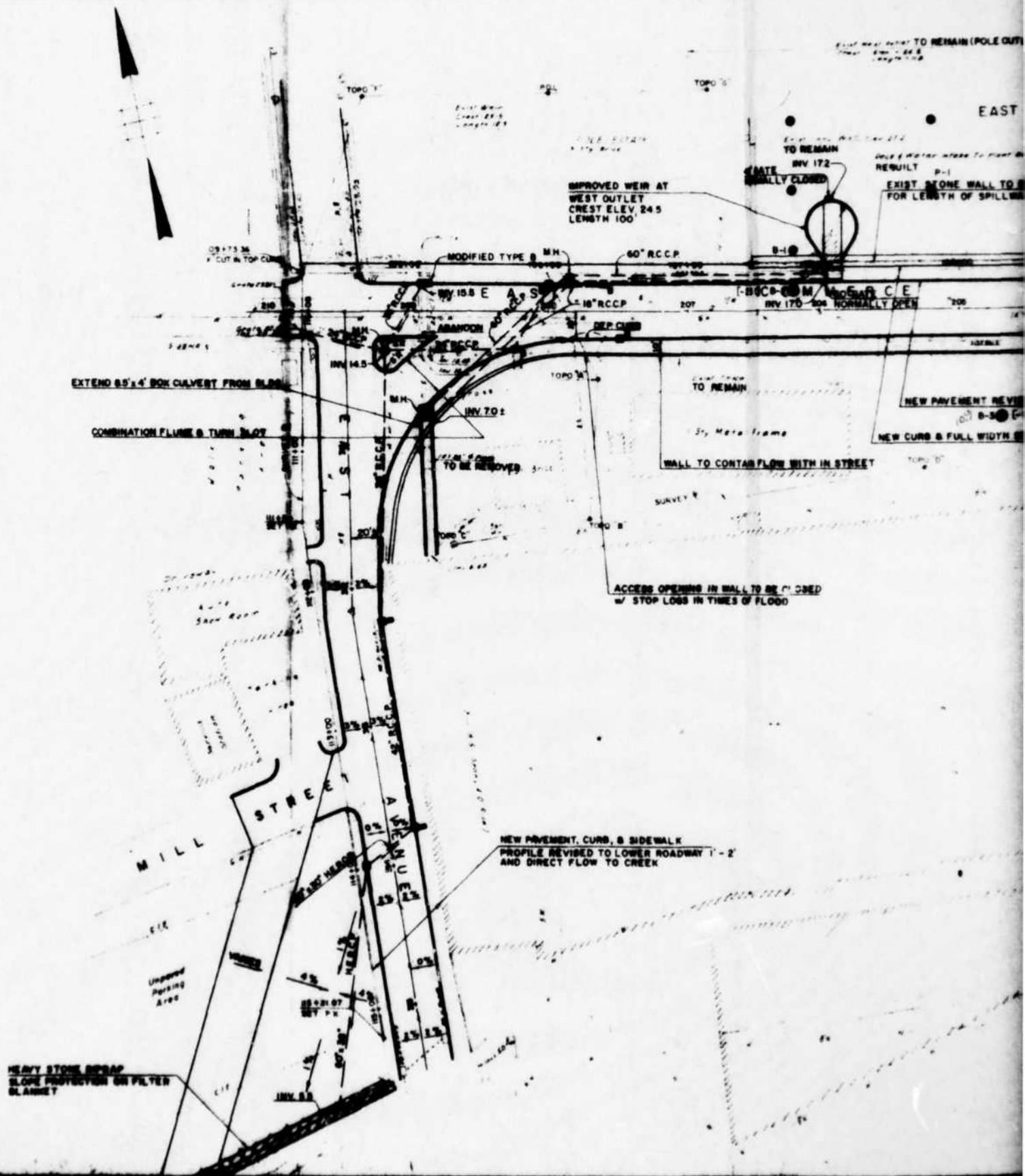
TO REMAIN (POLE OUTLET)

EAST LAKE ●P-2   ● ●P-4

TO REMAIN
INV 172   REBUILT   P-1
EXIST STONE WALL TO BE REMOVED
FOR LENGTH OF SPILLWAY   ●P-3   CREST EL 24.5
                                                 LENGTH 100'
            350' CONC SPILLWAY, CREST EL 26.0   REBUILT EXIST WALL
                                B-4   TO REMAIN
B-

       M A C E   STREET   SURVEY
INV 170   NORMALLY OPEN   205   204   205   202

                                              B-5
                                             -16.5
                                             -20.5

NEW PAVEMENT REVISED PROFILE
       (0) B-3 ● [-15.0]   B-6 ● [-15.0]

NEW CURB & FULL WIDTH SIDEWALK

WIDTH IN STREET

SOILS LEGEND

● PROBE
●P PROBE W/ BOTTOM SAMPLE
● 2½" BORING
● 2½" BORING W/ WELL POINT (depth)
● 2½" BORING W/ (2) WELL POINTS (depth)

PRELIMINARY
NOT APPROVED FOR CONSTRUCTION

HYDRAULIC DATA

DISCHARGE AT	ELEV. 26		EV. 27	
	EXIST	PROP		PROP
EAST OUTLET (PARTIALLY DEWATERED OPENS) (COMPLETE)	80/150	—		350
WEST OUTLET (LOOPAL OPENS)	0	250		350
POLE OUTLET	40	40		100
	0	0		1050
	80/250	490		1050

CUMBERLAND CO., CITY OF BRIDGETON

STUDY TO IMPROVE

CAPACITY & SAFETY OF

EAST LAKE & EAST COMMERCE ST. DAM

PLAN A₁

A.G. LICHTENSTEIN & ASSOC.
CONSULTING ENGINEERS
TRAVERSE, N.J.

SCALE 1"=40'

FIGURE 3

EAST AVE

PLANT ENTRANCE

EXISTING WALL & YARD

PROPOSED C

APPROX EXIST GROUND ALONG N R/W LINE

(-1.233%

LVC = 50'

FLUME & TURN SLOT
(TO EAST AVE)

EXISTING GROUND
PROPOSED GRADE

20.00		19.00		22.5		23.2		21.9		21.00		20.69	21.80 21.28

22.69 21.75   23.90 22.18   21.49 22.44   23.00 22.63

+50   212+0   +50   211+0   +50   210+0   +50   209+0   +50   208+0   +50   207+0   +50   206+0

206+20.91

EAST COMMERCE STREET PR

SCALE 1" = 40' HOR
1" = 4' VERT

C EAST COMMERCE ST

(+12.333%

FLUME & TURN SLOT
(TO COMMERCE ST)

EXISTING C PROFILE

-7.33%

23.65	23.16	22.80	20.0	19.0	17.7	16.77		14.14

+50   309+00   +40 +50   310+00   +18   +50   311+00   +32   +50   312+00   +32   +50   313+00
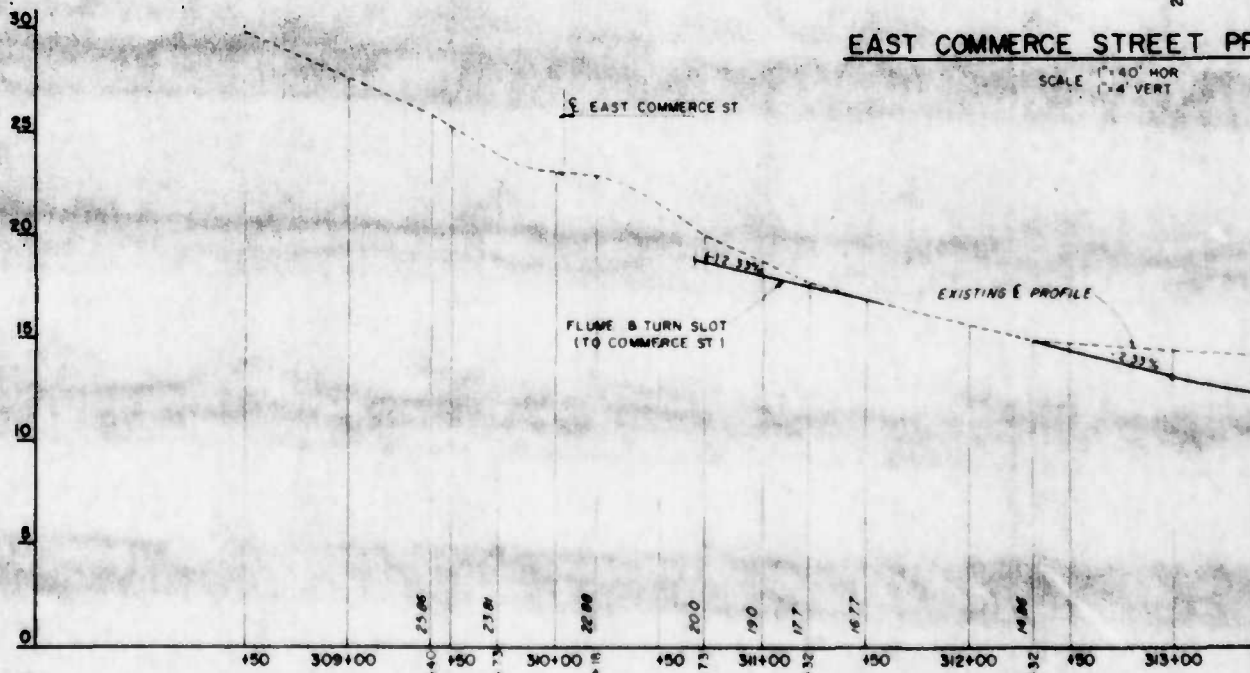
EAST AVENUE PROFILE

SCALE 1" = 40' HOR
1" = 4' VERT

LVC = 100'
PVI STA = 203+50
ELEV = 24.88
M.O. = 0.105

PGC STA 201+50

350' CREST LENGTH

TOP OF EXISTING STONE WALL
EXISTING ℄ PROFILE
ELEV 26.0
REBUILT STONE WALL
PROPOSED CREST

-1.74%

-0.95%

PROPOSED ℄ PROFILE

ERCE STREET PROFILE

SCALE 1"=40' HOR
1"=4' VERT

REVISED EAST AVE PROFILE
MEET EXIST PAV'T AT BRIDGE

-2.33%

(1) 10%

PARKING AREA TO
BE GRADED TO DRAIN
TO MILL CREEK

14.02

12.63

313+00   +50   314+00   +50   315+00   +2.07

FIGURE 4

| CUMBERLAND CO., CITY OF BRIDGETON |
| STUDY TO IMPROVE |
| CAPACITY & SAFETY OF |
| EAST LAKE & EAST COMMERCE ST. DAM |
| PROFILE |

| A.G. LICHTENSTEIN & ASSOC CONSULTING ENGINEERS TEANECK, N.J. | SCALE AS SHOWN | DATE MAR 78 | SHEET 3 OF 6 |

Note
Actual sluice geometry
from field notes supplied
by Olbert L. Frazinger

EMERGENC

Plan North

AVENUE

EAST

Building 4
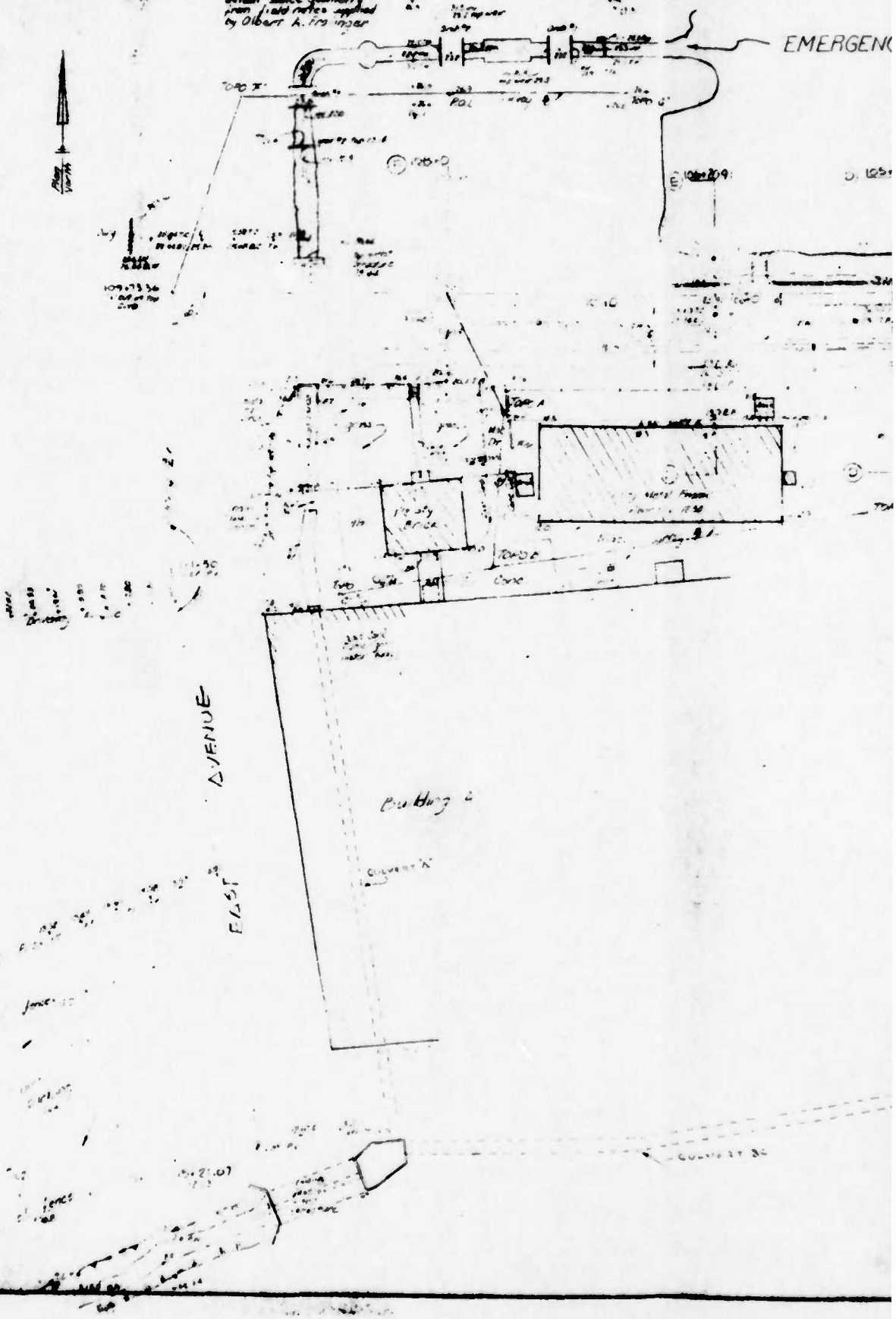
EMERGENCY SPILLWAY

EAST LAKE

PRINCIPAL SPILLWAY

EAST COMMERCE STREET

LEGEND

FIGURE 5

CUMBERLAND CO., CITY OF BRIDGETON		
STUDY TO IMPROVE		
CAPACITY & SAFETY OF		
EAST LAKE & EAST COMMERCE ST. DAM		
TOPOGRAPHY		
A.G. LICHTENSTEIN & ASSOC. CONSULTING ENGINEERS TEANECK, N.J.	SCALE 1"=40'	DATE MAR. 73	SHEET NO. 5 OF

Muck

Marsh

Muck

Less Than 1'

Muck

Marsh

Marsh

20.0

19.0 18.0 17.0

20.0 18.0

PRESENT SHORE LINE - ELEV. 21.2?
(WITH (2) 42" GATES OPEN)

OLD SHORE LINE - ELEV. 24.3?
(WITH (2) 42" GATES CLOSED)

MAXIMUM POOL ELEVATION - 27.2

NORMAL POOL ELEVATION - 24.2 (48 MILLION GALLONS)

PRESENT POOL ELEVATION - 21.2 (APR. '78)(28 MILLION GALLONS)

LAKE STAGE IN FEET (M.S.L.)

28
26
24
22
20
18
16
14
12
10

0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300

STORED VOLUME IN ACRE-FT. ———
SURFACE AREA IN SQUARE FEET X 10,000 —•—•—
ASSUMED SURFACE AREA PRIOR TO SOUNDINGS —×—×—

FIGURE 6

CUMBERLAND CO. CITY OF BRIDGETON		
STUDY TO IMPROVE		
CAPACITY & SAFETY OF		
EAST LAKE & EAST COMMERCE ST DAM		
TOPOGRAPHY		

| A G LICHTENSTEIN & ASSOC. CONSULTING ENGINEERS TEANECK , N J | SCALE AS NOTED | DATE MAY '76 | SHEET NO. 98 OF 98 |

GATED SPILLWAY

REGULATED OUTLET

WEIRS

CURB WALL

COMMERCE STREET

48" PIPES

BASIN

18" & 24" PIPES

SLUICE

FACTORY BLDGS.

CULVERT

CULVERT

BASIN

FIGURE 6A

SECTION D-D FROM FIGURES
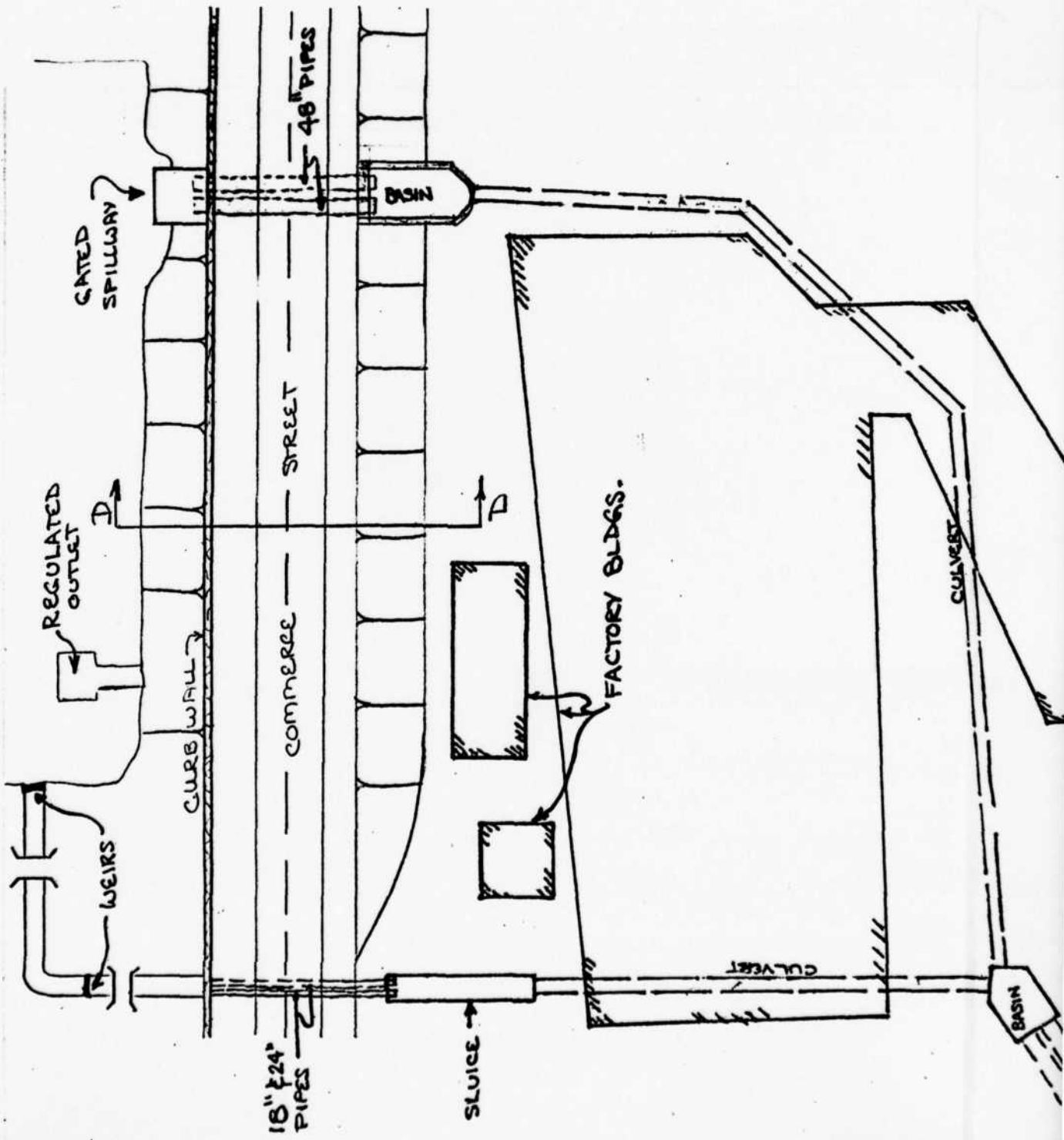
THOR ENGINEERS
341 SYLVAN AVENUE
LEONIA, NEW JERSEY 07605
201-944-5958

PROJECT East Commerce Street
Dam Study
LOCATION Bridgeton, NJ
NUMBER #7426

HOLE NO. B1
SHEET 1 OF 1
TYPE VERT. CASED
DATE 11-14-74

GROUND WATER OBSERVATION

At 10 ft. after 0 Hours

At ___ ft. after ___ Hours

	CASING	SAMPLER	CORE BAR
TYPE	Flush	55	
SIZE I.D.	3-1/2	1-3/8	
Hammer Wt.	300	148	BIT
Hammer Fall	18	30	

SURFACE ELEV. +22.3
DATUM USC & GS
BORING CONTR. Craig Testing
BORING FOREMAN M. Pratt
INSPECTOR L. Markowski

LOCATION:
West abutment at upstream toe of embankment (waters edge)

DEPTH BELOW SURFACE	CASING BLOWS PER FOOT	SAMPLE DEPTHS from/ to	NO. & TYPE of Sample	BLOWS PER 6" ON SAMPLER 18.2			RECOVERY	STRATA CHANGE DEPTH Elev.( )	FIELD DESCRIPTION	REMARKS
				0–6	6–12	12–18				
	1	0–	SS1	1	0	0	1	(+22.3	Tan br. very loose cmf SAND, tr. f gravel, silt	
	1	2								
	0	2–	SS2	1	0	1	1	4.0		
	1	4								
5	1	4–	SS3	1	1	0	1		Br. very loose cmf SAND, sm cmf gravel, tr silt	
	9	6						6.0		
	15	6–	SS4	2	6	13	14	(+16.3)		
	21	8							Br. very dense SAND, sm cmf gravel, tr silt	
	26	8–	SS5	13	19	24	29			
10	32	10								
	11	10–	SS6	10	11	14	6			
	9	12								
	12	12–	SS7	8	12	23	26			
	15	14								
15	20	14–	SS8	54	65	83		15.5		
								(+6.8)	Bottom of boring at 15.5 feet	

34

THOR ENGINEERS
341 SYLVAN AVENUE
LEONIA, NEW JERSEY 07605
201.944.5958

CLIENT _____
PROJECT East Commerce Street
Dam Study
LOCATION Bridgeton, NJ
NUMBER #7426

HOLE NO. B2
SHEET __1__ OF __
TYPE __HSA__
DATE __11/15/74__

GROUND WATER OBSERVATION			CASING	SAMPLER	CORE BAR
A: 1'0" ft after 24 Hours	TYPE	HSA	SS		
	SIZE I.D.	4"	1-3/8		
At 12.3 ft. after 25 Hours days	Hammer Wt.		140	BIT
	Hammer Fall		30	

SURFACE ELEV. +24.0
DATUM USC & GS
BORING CONTR. Craig Testing
BORING FOREMAN M. Pratt
INSPECTOR L. Markowski

LOCATION:
South of curb for crest roadway at west abutment

DEPTH BELOW SURFACE	CASING BLOWS PER FOOT	SAMPLE DEPTHS from/ to	NO. & TYPE of Sample	BLOWS PER 6" ON SAMPLER 0–6	6–12	12–18	RECOVERY	STRATA CHANGE DEPTH Elev.( )	FIELD DESCRIPTION	REMARKS
	–	0–1		Drilled				1.0	4-1/2" Asphalt	
		1–	SS1	3	3	9	12	(+23.0)		
		3								
5		3–	SS2	10	9	8	8		Bt tan med dense to	
		5							dense cmf SAND &	
		5–7	SS3	8	9	5	3		cmf GRAVEL, tr silt	
		7–9	SS4	2	2	1	2			
10		9–11	SS5	15	22	14	12			
		11–13	SS6	10	20	14	13	13.0		
		13–15	SS7	10	7	6	5		Bryel med dense	
15								15.0	SAND, sm mf gravel, tr silt, org. m	
		15–17	SS8	6	10	16	29	(+9.0)		
		17–19	SS9	13	36	55	74		Br yel gy very dense cmf SAND, trf gravel, silt	
20										
								22.0 (+2.0)	Br gy very dense cmf SAND, sm mf gravel	
25								26.5		
								(-1.5)	Bottom of boring at 26.5 feet	Observation well installed at 25. feet
30										

FIGURE 8

THOR ENGINEERS
341 SYLVAN AVENUE
LEONIA, NEW JERSEY 07605
201.944.5958

CLIENT.
PROJECT East Commerce Street
Dam Study
LOCATION Bridgeton, NJ
NUMBER #7426

GROUND WATER OBSERVATION

At 1.5 ft. after 0 Hours

At 2.1 ft. after _____ Hours
25 days

TYPE	CASING HSA	SAMPLER SS	CORE BAR
SIZE I.D.	3-1/2"	1-3/8"	
Hammer Wt.		140	BIT
Hammer Fall		30	

SURFACE ELEV. +13.0
DATUM USC & GS
BORING CONTR. Craig Testing
BORING FOREMAN M. Pratt
INSPECTOR L. Markowski

LOCATION:
Downstream toe of slope – west abutment

DEPTH BELOW SURFACE	CASING BLOWS PER FOOT	SAMPLE DEPTHS from/ to	NO. & TYPE of Sample	BLOWS PER 6" ON SAMPLER 0-6	6-12	12-18	RECOVERY	STRATA CHANGE DEPTH Elev.( )	FIELD DESCRIPTION	REMARKS
0		0-2	S-1	4	9	12	13	(+13.0)	Br yel med dense	
		2-4	S-2	4	5	8	9	3.5	cmf SAND, tr/f gravel, tr silt	
5		4-6	S-3	15	22	22	20	(+9.5)	Br very dense SAND&	
		6-8	S-4	28	39	50	64	8.0	cmf GRAVEL, tr silt	
		8-10	S-5	8	18	23	37	(+5.0)		
10									Br very dense cmf SAND, tr silt, tr gravel	
15								15.0		
								(+2.0)	Bottom of boring at 15.0 feet	Observation well 15.0 feet

FIGURE 9

THOR ENGINEERS
341 SYLVAN AVENUE
LEONIA, NEW JERSEY 07605
201.944.5958

CLIENT _____
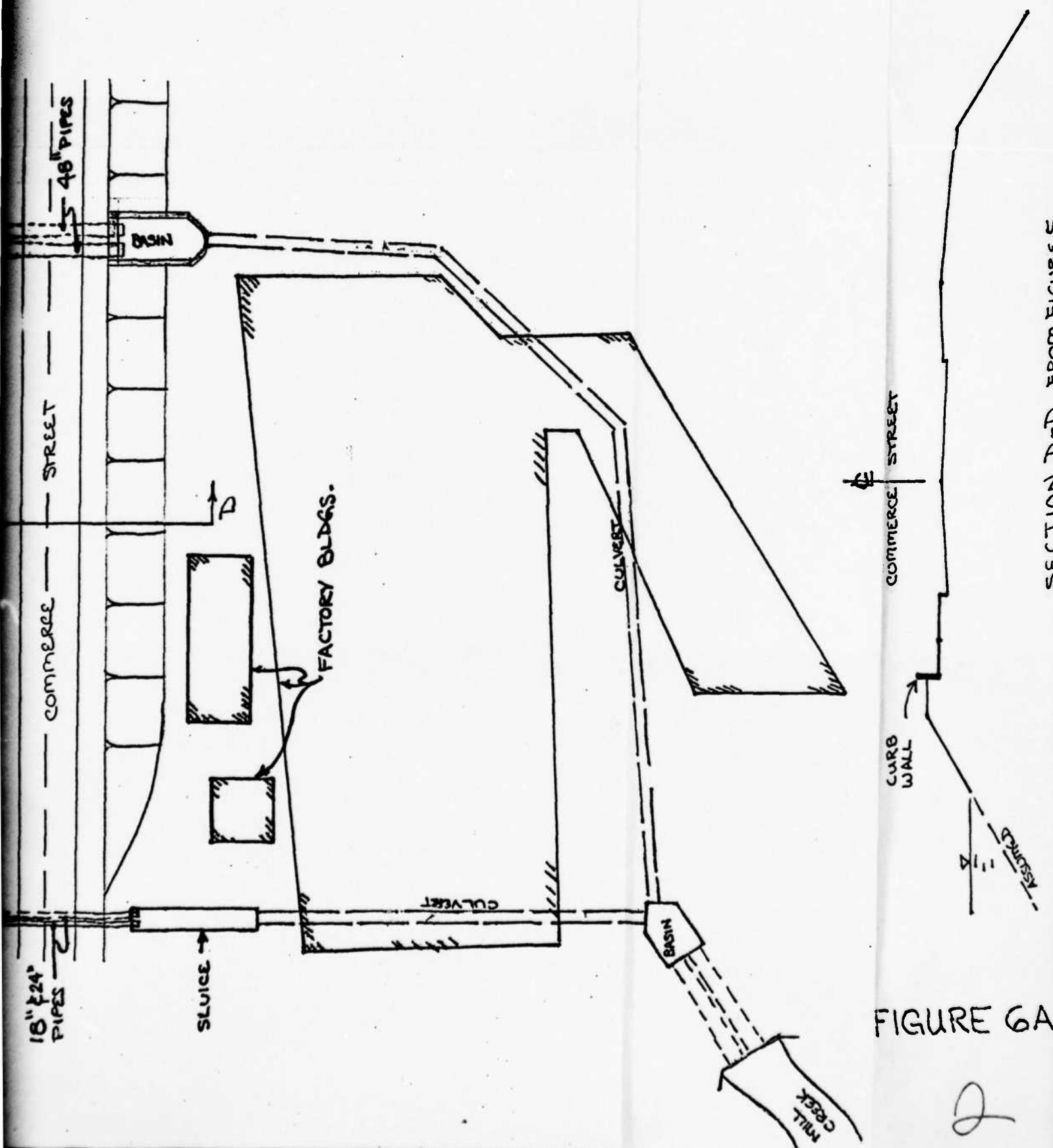PROJECT East Commerce Street
Dam Study
LOCATION Bridgeton, NJ
NUMBER #7426

HOLE NO. B-1
SHEET 1 OF
TYPE VERT CAS
DATE 11-13-

GROUND WATER OBSERVATION				
	TYPE	CASING	SAMPLER SS	CORE BAR
A: 0.5 ft. after 0 Hours	SIZE I.D.	2-1/2	1-3/8	
	Hammer Wt.	300#	140#	BIT
At _____ ft. after _____ Hours	Hammer Fall	24"	30"	

SURFACE ELEV. +21.8
DATUM USC & GS
BORING CONTR. Craig Test
BORING FOREMAN M. Prat
INSPECTOR T. Otto

LOCATION:
Middle of embankment - upstream toe of slope

DEPTH BELOW SURFACE	CASING BLOWS PER FOOT	SAMPLE DEPTHS from/ to	NO. & TYPE of Sample	BLOWS PER 6" ON SAMPLER			RECOVERY	STRATA CHANGE DEPTH Elev.( )	FIELD DESCRIPTION	REMARKS
				0—6	6—12	12—18				
0		0-2	1	1	0	0	0	(+21.8)	Tan br very loose	
		2-4	2	1	0	0	1		cmf SAND, tr mf gravel, tr silt	
5		4-6	3	1	1	0	1	6.0		
		6-8	4	1	1	2	1	(+15.8)	Br very loose cmf SAND & mf GRAVEL,	
		8-10	5	1	1	1	6		tr silt	
10		10-12	6	3	1	1	1	12.0		
		12-14	7	Pushed				13.5 (+5.3)	Br bl gy PEAT & ORGANIC SILT (fi	
15		14-16	8	1	3	7	16		Br gy med dense cmf SAND & mf GRAVEL	
								18.5 (+3.3)		
20		20-22	9	14	12	21	12		Tan ye br dense cmf SAND, tr/f gravel, tr silt	
								22.0 (-0.2)	Bottom of boring at 22.0 feet	
25										
30										
35										

FIGURE

THOR ENGINEERS			
341 SYLVAN AVENUE	PROJECT East Commerce Street		HOLE NO. DD
LEONIA, NEW JERSEY 07605	Dam Study		SHEET 1 OF 1
201.944.5958	LOCATION Bridgeton, NJ		TYPE VERT HSA
	NUMBER #7426		DATE 11/13/74

GROUND WATER OBSERVATION		CASING	SAMPLER	CORE BAR	SURFACE ELEV. +25.5
A: 15.0 ft. after 0 Hours	TYPE	HSA	SS		DATUM USC & GS
	SIZE I.D.	4"	1-3/8		BORING CONTR. Craig Testing
At 14.1 ft. after ___ Hours	Hammer Wt.		140#	BIT	BORING FOREMAN M. Pratt
25 days	Hammer Fall		30"		INSPECTOR L. Markowski

LOCATION:

Middle of south sidewalk - middle of embankment

DEPTH BELOW SURFACE	CASING BLOWS PER FOOT	SAMPLE DEPTHS from/ to	NO. & TYPE of Sample	BLOWS PER 6" ON SAMPLER 18x2 0-6	6-12	12-18	RECOVERY	STRATA CHANGE DEPTH Elev.( )	FIELD DESCRIPTION	REMARKS
		0-1.3	S1A					(+25.5) 1.3	Concrete sidewalk slab and base	
		2-4	S1B	5	6	5	5	(+24.0) 2.5	Bl br ORG material, cinders Bl br soft silty CLAY	cinders
5		4-6	S2	5	6	6	6	(+21.5) 6.0	Br med dense mf SAND & CINDERS, org material	
		6-8	S3	4	5	3	2	(19.5) 8.0	Br bl loose SAND, tr clay, tr silt, org matl	
10		8-10	S4		Pushed			(17.5)	Bl br org mtl, ashes cinders	
		10-12	S5	1	1	1	1	12.0		
		12-14	S6	2	3	3	4	(+13.5)	Br bl org mtl, ashes wood chips, sawdust	
15		14-16	S7A	5	28	17	11	15.0		
			S7B					16.0	Gy cmf SAND, wood	
		16-18	S8	10	12	8	3	(+9.5) 18.0	Gy br med dense cmf SAND, tr	
20		18-20	S9	12	23	24	26	(+7.5)	Gy br dense cmf SAND, tr silt (org mtl interbedded)	
								22.0		
25								(+3.5)	Br very dense cmf SAND, sm to tr mf gravel	
		25-26	S10	18	47	71				
30		30-31	S11	8	17	29				
35		35-36	S12	122				35.5		
								(-10.0)	Bottom of Boring at 35.5 feet	Well points installed at 15.5 feet and 25.5 feet depths.

FIGURE 11

THOR ENGINEERS

THOR ENGINEERS
341 SYLVAN AVENUE
LEONIA, NEW JERSEY 07605
201.944.5958

CLIENT _____
PROJECT East Commerce St. Dam
            Study
LOCATION Bridgeton, NJ
NUMBER #7426

GROUND WATER OBSERVATION		CASING	SAMPLER	CORE BAR
At 2" ftX after 0 Hours	TYPE	Flush	SS	
	SIZE I.D.	2-1/2"	1-3/8"	
At ___ ft. after ___ Hours	Hammer Wt.	300#	140#	BIT
	Hammer Fall	24"	30"	

SURFACE ELEV. 21.4
DATUM USC & GS
BORING CONTR. Craig Testin
BORING FOREMAN M. Pratt
INSPECTOR T. Otto

LOCATION:
Upstream toe of slope east abutment adjacent to spillway structure.

DEPTH BELOW SURFACE	CASING BLOWS PER FOOT	SAMPLE DEPTHS from/ to	NO. & TYPE of Sample	BLOWS PER 6" ON SAMPLER 0-6	6-12	12-18	RECOVERY	STRATA CHANGE DEPTH Elev.( )	FIELD DESCRIPTION	REMARKS
0		0-2	SS1	2	3	1		(+21.4	Br loose cmf SAND,	
								2.0	sm mf gravel	
		2-4	SS2	2	1	1	1	(+19.4	Dk br gy very loose	
5		4-6	SS3	1	1	1	1		cmf SAND, org mtl	
									interbedded	
		6-8	SS4	1	0	0	1			
		8-10	SS5	1	0	0	1	9.0		
10								10.5	Dk gy soft cmf SAND & ORG SILT	
		10-12	SS6	1	1	0	1	12.0	Br gy loose org silty cmf SAND	
		12-14	SS7	1	2	2	8	13.5	Dk br PEAT & ORG SILT	
								14.	PEAT & WOOD fiber	
15		14-16	SS8	9	13	11	9	(+7.4	By br med dense cmf SAND & cmf GRAVEL	
20								20		
		25-26	S10	5	9	17		(+1.4	Tan ye br dense cmf SAND	
25								26.5		
								(-5.1)	Bottom of boring at 26.5 feet	
30										
35										

FIGURE 12

THOR ENGINEERS
741 SYLVAN AVENUE
LEONIA, NEW JERSEY 07605
201.244.5958

PROJECT __East Commerce Street__
__Dam Study__
LOCATION __Bridgeton, NJ__
NUMBER __#7426__

GROUND WATER OBSERVATION				CASING	SAMPLER	CORE BAR		
At __16.5__ ft. after __24__ Hours	TYPE	HSA	SS		SURFACE ELEV. __+27.7__			
	SIZE I.D.	4"	1-3/8		DATUM __USC & GS__			
At __14.1__ ft. after ____ Hours	Hammer Wt.		140#	BIT	BORING CONTR. __Craig Testing__			
__25 days__	Hammer Fall		30"		BORING FOREMAN __M. Pratt__			
					INSPECTOR __L. Markowski__			

LOCATION:
North curb for crest roadway at east abutment

DEPTH BELOW SURFACE	CASING BLOWS PER FOOT	SAMPLE DEPTHS from/ to	NO. & TYPE of Sample	BLOWS PER 6" ON SAMPLER			RECOVERY	STRATA CHANGE DEPTH Elev.( )	FIELD DESCRIPTION	REMARKS
				0—6	6—12	12—18				
0		0-2	S1	8	9	5	8	(+27.7) 2.0	Bl br dense cmf SAND & GRAVEL, cinders, wood, brick	
		2-4	S2	12	9	5	4	(+25.7)	Tan bn loose cmf SAND, tr silt tr clay, wood fibers	
5		4-6	S3	2	2	3	3	6.0		
		6-8	S4	7	3	4	4	(+21.7)	Tan loose cmf SAND, tr silt, clay f gravel	
		8-10	S5	7	4	3	3	10.0		
10		10-12	S6	4	3	3	2	(+17.7)	Br tan loose cmf SAND, tr silt, clay, f gravel	
		12-14	S7	4	2	2	2			
15		14-16	S8	3	2	2	3			
		16-18	S9	2	1	3	5	18.0		
		18-20	S10	3	3	9	21	(+9.7)	Gy br dense cmf gravelly SAND	
20		20-22	S11	25	43	37	27	23.0		
25		25-26.5	S12	37	56	69		(+4.7) 26.5	By br very dense cmf SAND, sm mf gravel	
								(+1.2)	Bottom of boring at 26.5 feet	Observation well installed at 25.0 feet
30										
35										

FIGURE 13

GROUND SURFACE TO __25__ FT. USED HSA

THOR ENGINEERS
041 SYLVAN AVENUE
LEONIA, NEW JERSEY 07605
211.944.5038

CLIENT	
PROJECT	East Commerce Street Dam Study
LOCATION	Bridgeton, NJ
NUMBER	#7426

HOLE NO.	B9
SHEET	1 OF 1
TYPE	VERT, HSA
DATE	11/15/74

GROUND WATER OBSERVATION

At ___ ft. after ___ Hours

At 3.5 ft. after ___ Hours
25 days

	CASING HSA	SAMPLER SS	CORE BAR
TYPE			
SIZE I.D.	4"	1-3/8"	
Hammer Wt.		140#	BIT
Hammer Fall		30#	

SURFACE ELEV. +17.7
DATUM USC & GS
BORING CONTR. Craig Testing
BORING FOREMAN M. Pratt
INSPECTOR L. Markowski

LOCATION: Downstream toe of embankment adjacent to spillway and east abutment.

DEPTH BELOW SURFACE	CASING BLOWS PER FOOT	SAMPLE DEPTHS from/ to	NO. & TYPE of Sample	BLOWS PER 6" ON SAMPLER 0—6	6—12	12—18	RECOVERY	STRATA CHANGE DEPTH Elev.( )	FIELD DESCRIPTION	REMARKS
		0-2	S1	6	8	8	9	(+17.7) 2.0	Bl br med dense cmf SAND & cmf GRAVEL, cinders	
		2-4	S2A S2B	6	8	6	12	3.5	Tan br med dense cmf SAND, sm mf gravel tr clay, peat	
5		4-6	3	7	6	4	7	(+14.2) 6.0	Tan br med dense cmf SAND, tr mf gravel, silt, clay	
		6-8	4	1	2	10	11	8.0	Br dense cmf SAND, sm mf gravel, tr clay	
		8-10	5	8	12	11	13	(+9.7)	Br tan dense cmf SAND, tr mf gravel	
10										
15								15.0		
								(+2.7)	Bottom of boring at 15.0 feet	Well observation installed at 15.0 feet
20										
25										

FIGURE 14

SCALE 1" = 4 MILES

LEGEND:

[Cohansey Sand pattern]	COHANSEY SAND	QUARTZ SAND.
[Kirkwood Sand pattern]	KIRKWOOD SAND	FINE MICACEOUS SAND.
[Bridgeton Formation pattern]	BRIDGETON FORMATION	GRAVEL AND SAND.
[Cape May Formation pattern]	CAPE MAY FORMATION	GRAVEL AND SAND.

FIGURE 15
GEOLOGIC MAP

APPENDIX

FIELD INSPECTION REPORT

Check List
Visual Inspection
Phase 1

Name Dam __East Lake Dam__   County __Cumberland__   State __New Jersey__   Coordinators __New Jersey DEP__

Mr. Larry Woscyn.

Date(s) Inspection __2/3/78__   Weather __Clear__   Temperature __30's__

Pool Elevation at Time of Inspection __171__ M.S.L.   Tailwater at Time of Inspection __---__ M.S.L.

Inspection Personnel:

Mr. Lee DeHeer   Mr. Gurbaksh Sanghera
_____   _____

Mr. George Elias   _____
_____

Mr. Stefan Manea   Mr. Gurbaksh Sanghera __Recorder__
_____   _____

Accompanied by:

Mr. Larry Woscyna, New Jersey Department of Enviromental Protection

Mr. Robert Plummer, Plant Engineer, Murbeck Fabrics

Mr. Burt Diamond, Plant Manager, Murbeck Fabrics

A1

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS	NONE NOTED.	NONE
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	NONE NOTED	NONE
SLOUGHING OR EROSION OF EMBANKMENT AND ABUTMENT SLOPES	At several locations, bank sloughing has created vertical scarps of 2-3 feet on the upstream face. Settlement across the top of dam was noted at several locations with one settlement area of about 120 sq.feet with a maximum settlement of 2.5 feet.	The upstream face should be repaired with a suitable material and provided with an appropriate riprap surface.
VERTICAL AND HORIZONTAL ALINEMENT OF THE CREST	The stone and masonry wall across the top of the dam is presently misaligned, both horizontally and vertically, revealing differential movement of the embankment.	
RIPRAP FAILURES	Upstream face is not riprapped.	Face should be covered with heavy stone to protect the embankment.

A2

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
JUNCTION OF EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM	Embankment erosion was noted to depths of 3-4 feet on both sides of the east outlet structure for a length of 30-50 feet.	These areas should be replaced with suitable materials and protected by training walls or heavy riprap.
ANY NOTICEABLE SEEPAGE	Seepage occurs along portions of the downstream face following periods of rainfall excess. The seepage appears to be due to inadequate storm drainage causing ponding along East Commerce St. and seems to be related to the settlement areas noted on the previous page.	Seepage might be controlled or eliminated through installation of adequate storm drainage facilities along the road atop the dam.
STAFF GAGE AND RECORDER	No gages are in use at this site.	NONE
DRAINS	French drains were installed at the toe of the embankment in 1955. Water accumulating in the drains is pumped out. According to Bob Plummer, Junior Plant Engineer at the Bridgeton Dyeing and Finishing Co., the drains maintain a ground water level about 6-inches below the floor of the plant.	NONE

OUTLET WORKS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	The outlet conduit consists of two 48-inch cast iron pipes lined at their entrances with 42-inch corrugated metal pipes. The pipes are buried and no cracking or spalling was noted at exposed locations.	NONE
INTAKE STRUCTURE	The east outlet intake structure is badly deteriorated, with spalling at a number of locations. The columns are cracked and a large crack is located near the junction of the beam with the slab.	The intake appears to be deficient both structurally and hydraulically. Repair or replacement should be effected as merited.
OUTLET STRUCTURE	The two 48-inch outlet conduits are supported by a stone and masonry wall at the downstream face of the dam. Discharge falls into a concrete basin located about 5-feet below the pipe inverts.	No problems were noted.
OUTLET CHANNEL	The outlet channel is a box culvert 5.8 feet wide by 5.66 feet high carrying discharge about 900-feet from the concrete basin to Mill Creek.	No problems were noted.
EMERGENCY GATE	The intake structure is equipped with 48-inch circular gates. The gates are badly rusted and due to the questionable safety of this project, New Jersey DEP, in 1973, ordered that the gates be kept open at all times.	See remarks under "Intake Structure"

A4

RESERVOIR

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SLOPES	Flat slopes, no signs of sliding	NONE
SEDIMENTATION	The reservoir appears to be moderately silted.	Siltation does not affect the stability of the dam.

DOWNSTREAM CHANNEL

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION (OBSTRUCTIONS, DEBRIS, ETC.)	Flow from the dam to the down stream channel must travel through various open channels and culverts. The stream is subject to tidal variations that affect the discharge capacity of the outlet works.	NONE
SLOPES	No problems were observed.	NONE
APPROXIMATE NO. OF HOMES AND POPULATION	The area downstream of the dam consists of business and industry with some residential areas (approximately 1,000 people).	

ITEM	REMARKS
MONITORING SYSTEMS	NONE
MODIFICATIONS	NONE
HIGH POOL RECORDS	No official records for this site. Unofficial information on flooding during the tropical storm of 1972 is included in Section 1.3.b.
POST CONSTRUCTION ENGINEERING STUDIES AND REPORTS	A previous study, including evaluation and proposed improvements to the dam outlet works, was completed by A.G. Lichtenstein and Associates, Consulting Engineers, Teaneck, NJ (dated August, 1976).
PRIOR ACCIDENTS OR FAILURE OF DAM DESCRIPTION REPORTS	No information available.
MAINTENANCE OPERATION RECORDS	No records available.

A7

ITEM	REMARKS
DESIGN REPORTS	The dam is reported to have been built in 1805. No records available.
GEOLOGY REPORTS	No information available.
DESIGN COMPUTATIONS HYDROLOGY & HYDRAULICS DAM STABILITY SEEPAGE STUDIES	½ PMF - Inflow peak 12,500 cfs; routed outflow peak 12,450 cfs (29.9 feet above MSL). This discharge overtops the dam. The embankment is in poor condition. No seepage studies were made.
MATERIALS INVESTIGATIONS BORING RECORDS LABORATORY FIELD	Logs of eight soil borings (November, 1974) along the embankment are shown on Figures 7 through 14. The boring locations are shown on Figure 2.
POST-CONSTRUCTION SURVEYS OF DAM	See Figures 5 and 6
BORROW SOURCES.	Unknown.

PHOTOGRAPHS

RESERVOIR OUTLET STRUCTURE AT EAST END
OF EMBANKMENT
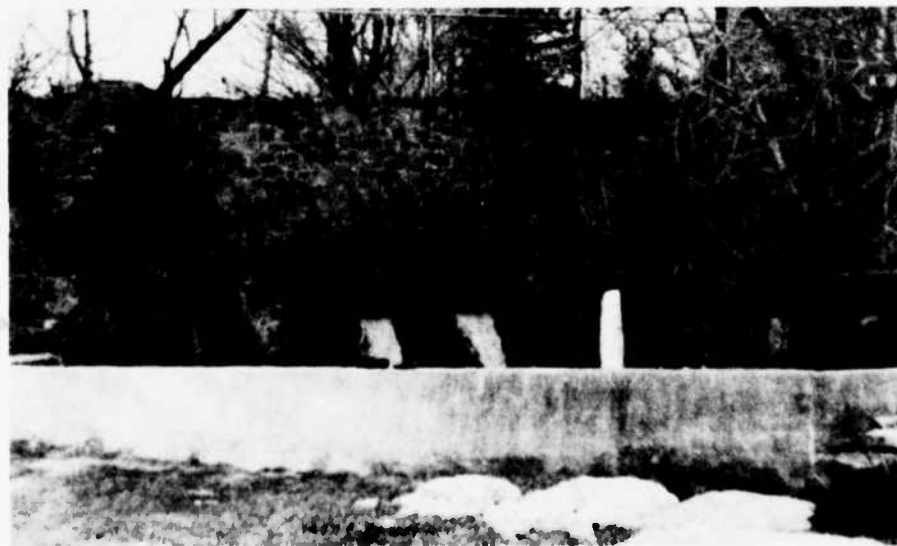
OUTFALL FROM RESERVOIR OUTLET STRUCTURE

BOX CULVERT DOWNSTREAM FROM OUTFALL
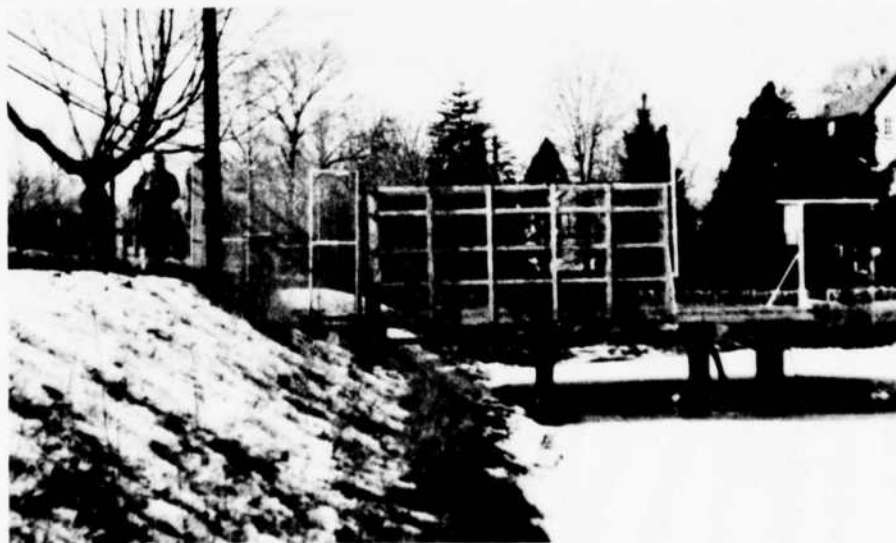
INTAKE STRUCTURE FOR INDUSTRIAL PLANT

SPILLWAY ON WEST SIDE OF RESERVOIR

SPILLWAY CHANNEL LOOKING DOWNSTREAM

A11

TOP OF DAM LOOKING EAST—DOWNSTREAM SIDE

DOWNSTREAM FACE OF EMBANKMENT LOOKING WEST

A12

HYDRAULIC AND HYDROLOGIC

CALCULATIONS

NAME OF CLIENT CORPS OF ENGINEERS

PROJECT EAST LAKE DAM

PROBABLE MAXIMUM FLOOD CALCULATION

DRAINAGE AREAS

JACKSON RUN BRANCH — 1.36 SQ. MI.
INDIAN FIELDS BRANCH — 5.90 SQ. MI.

7.26 SQ. MI.

FROM HYDROMETEOROLOGICAL REPORT #33

6 HOUR — 10 SQ. MI. PMP = 27"   ZONE #6

SINCE THE DRAINAGE AREA IS UNDER 10 SQ. MI., NO REDUCTION REFLECTING BASIN SIZE IS USED.

6 HR. PMP	1.00 × 27" =	27.0"
12 HR. PMP	1.09 × 27" =	29.4"
24 HR. PMP	1.17 × 27" =	31.6"
48 HR. PMP	1.26 × 27" =	34.0"

PMP VALUES ARE REDUCED 20% FOR PROBABLE MISALIGNMENT OF BASIN & STORM ISOHYETALS.

6 HR PMP — 21.6"
12 HR PMP — 23.5"   (23.5 − 21.6)/6 = 1.9/6 = .32"/HR   ✱
24 HR PMP — 25.3"   (25.3 − 23.5)/6 = 1.8/12 = .15"/HR   ✱
48 HR PMP — 27.2"   (27.2 − 25.3)/6 = 1.9/24 = .08"/HR   ✱

✱ THESE PRECIPITATION INCREMENTS CAN BE NEGLECTED.

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PROJECT EAST LAKE DAM

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ESTIMATION OF T_c

BUREAU OF PUBLIC ROADS

JACKSON RUN BRANCH

$$T_c = \left(11.9\, L^3 / H\right)^{.385}$$ L = 2.69 MILES   H = 110' - 25' = 85'

$$T_c = \underline{1.5}\ \text{HOURS}$$

INDIAN FIELDS BRANCH

$$T_c = \left(11.9\, L^3 / H\right)^{.385}$$ L = 4.02 MILES   H = 110' - 25' = 85'

$$T_c = \underline{\underline{2.34\ \text{HOURS}}}$$

UPLAND METHOD (SCS)

JACKSON RUN BRANCH

① AVERAGE LAND SLOPE ≈ 2.0%   UPLAND LENGTH (L_u) = 7000

VELOCITY = 1.5 fps (NEARLY BARE GROUND)

T_{t_1} = 7000 / 1.5 = 4667 sec = 78 min = 1.3 hours

② STREAM LENGTH (L_s) = 7200'

$$V_{avc} \approx \frac{1.49}{.04} \times 3^{2/3} \times .005^{1/2} \approx 5.5\ \text{fps}$$

T_{t_2} = 7200 / 5.5 = 1309 sec = 22 min = .36 hours

$$T_c = 1.3 + .36 = \underline{1.66}\ \text{hours}$$

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INDIAN FIELDS BRANCH

①

AVERAGE LAND SLOPE $\approx 0.75\%$ UPLAND LENGTH $(L_o) = 52$

VELOCITY $\approx .85$ fps (NEARLY BARE GROUND)

$T_{t_1} = 5200/.85 = 6118$ sec ≈ 1.70 hours

②

STREAM LENGTH $(L_s) = 16000'$

$V_{ave} = \dfrac{1.49}{.04} \times 3^{2/3} \times .0026^{1/2} = 3.95$ fps

$T_{t_2} = 16000/3.95 = 4050$ sec ≈ 1.13 hours

$T_c = 1.70 + 1.13 = \underline{2.83}$ hours.

UNIT HYDROGRAPH COMPUTATION (USING BPR VALUES FOR T_c)

JACKSON RUN BRANCH

$T_p = \dfrac{D}{2} + .6 T_c$ $(D = .5 \text{ HR})$

$T_p = .25 + .6 \times 1.5 = 1.15 \text{ HR}.$

$T_b = 2.67 \times T_p = 2.9 \text{ HR}.$

$q_p = 484 A / T_p = 484 \times 1.36 / 1.1 = \textbf{598 cfs}$

INDIAN FIELDS BRANCH

$T_p = \dfrac{D}{2} + .6 \times T_c$ $(D = .5 \text{ HR})$

$T_p = .25 + .6 \times 2.3 = 1.63 \text{ HR}.$

$T_b = 2.67 \times T_p = 4.35 \text{ HR}.$

$q_p = 484 A / T_p = 484 \times 5.9 / 1.63 = 1752$

"PMP" RAINFALL INCREMENTS (MAX. 6 HRS.) D=.5HR.

TIME (HRS)	Σ% 6HR.PMP	Σ 6HR. PMP	INCREMENTAL Pm
0.5	30	6.5"	6.5"
1.0	49	10.6	4.1
1.5	58	12.5	1.9
2.0	65	14.0	1.5
2.5	71	15.3	1.3
3.0	76	16.4	1.1
3.5	80	17.3	.9
4.0	84	18.1	.8
4.5	88	19.0	.9
5.0	92	19.9	.9
5.5	96	20.7	.8
6.0	100	21.6	.9

ADJUSTED 6 HR. SEQUENCE FOR A THIRD QUARTILE PEAK RUNOFF

TIME (HRS.)	INCREMENTAL PMP	Σ 6 HR. PMP
.5	.8	.8
1.0	.8	1.6
1.5	.9	2.5
2.0	.9	3.4
2.5	1.1	4.5
3.0	1.3	5.8
3.5	4.1	9.9
4.0	6.5	16.4
4.5	1.9	18.3
5.0	1.5	19.8
5.5	.9	20.7
6.0	.9	21.6

21.6 ✓

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NAME OF CLIENT CORPS OF ENGINEERS

PROJECT EAST LAKE DAM

SOIL COMPLEX NUMBER

BRIDGETON SOILS COMMONLY SANDS & GRAVELS, WITH SOME LOCAL SILTS AND CLAYS (SOIL GROUP B).

JACKSON RUN BRANCH

≃ 50% COMMERCIAL & BUSINESS   CN 92
≃ 30% RESIDENTIAL   CN 75
≃ 10% MEADOWS   CN 58
≃ 10% WOODS   CN 60

WEIGHTED CN:

$$.5 \times 92 + .3 \times 75 + .1 \times 58 + .1 \times 60 = \underline{80}$$

INDIAN FIELDS BRANCH

≃ 15% COMMERCIAL & BUSINESS   CN 92
≃ 20% RESIDENTIAL   CN 75
≃ 35% MEADOWS   CN 58
≃ 25% WOODS   CN 60
≃ 5% SWAMP   CN 85

WEIGHTED CN:

$$.15 \times 92 + .2 \times 75 + .35 \times 58 + .25 \times 60 + .05 \times 85 = \underline{68}$$

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PROJECT EAST LAKE DAM

JACKSON RUN BRANCH — DIRECT RUNOFF COMPUTATIONS

TIME (HR)	PMP RAINFALL INCR	PMP RAINFALL Σ	DIRECT RUNOFF INCR.	DIRECT RUNOFF Σ	LOSSES INCR.	LOSSES Σ
.5	.8	.8	0.0	.0.0	.8	.8
1.0	.8	1.6	0.3	0.3	.5	1.3
1.5	.9	2.5	0.6	0.9	.3	1.6
2.0	.9	3.4	.7	1.6	.2	1.8
2.5	1.1	4.5	.9	2.5	.2	2.0
3.0	1.3	5.8	1.1	3.6	.2	2.2
3.5	4.1	9.9	3.8	7.4	.3	2.5
4.0	6.5	16.4	6.3	13.7	.2	2.7
4.5	1.9	18.3	1.8	15.5	.1	2.8 *
5.0	1.5	19.8	1.4	16.9	.1	2.9 *
5.5	.9	20.7	.8	17.7	.1	3.0 *
6.0	.9	21.6	.8	18.5	.1	3.1

* MINIMUM LOSS RATE .1"/ HALF HOUR

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PROJECT EAST LAKE DAM

INDIAN FIELDE BRANCH - DIRECT RUNOFF COMPUTATIONS

TIME (HR)	PMP RAINFALL		DIRECT RUNOFF		LOSSES	
	INCR	Σ	INCR	Σ	INCR	Σ
.5	.8	.8	0.0	0.0	0.8	0.8
1.0	.8	1.6	0.1	0.1	0.7	1.5
1.5	.9	2.5	0.3	0.4	0.6	2.1
2.0	.9	3.4	0.4	0.8	0.5	2.6
2.5	1.1	4.5	0.7	1.5	0.4	3.0
3.0	1.3	5.8	1.0	2.5	0.3	3.3
3.5	4.1	9.9	3.4	5.9	0.7	4.0
4.0	6.5	16.4	6.0	11.9	0.5	4.5 *
4.5	1.9	18.3	1.8	13.7	0.1	4.6 *
5.0	1.5	19.8	1.4	15.1	0.1	4.7 *
5.5	.9	20.7	.8	15.9	0.1	4.8 *
6.0	.9	21.6	.8	16.7	0.1	4.9

* MINIMUM LOSS RATE .1" / HALF HOUR

SHEET NO. _9_ OF _____
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NAME OF CLIENT: CORPS OF ENGINEERS

PROJECT: EAST LAKE DAM

UNIT HYDROGRAPHS

JACKSON RUN BRANCH

$T_p = 1.15$ HOURS $q_p = 598$ cfs

T/TP	q/qp	TIME(HRS)	DISCHARGE(CFS)	ADJUSTED q's FOR 1" RUNOFF
0.0	0.00	0.0	0	0
0.1	0.015	0.12	9	8
0.2	0.075	0.23	45	42
0.3	0.16	0.35	96	90
0.4	0.28	0.46	167	157
0.5	0.43	0.58	257	242
0.6	0.60	0.69	359	338
0.7	0.77	0.81	460	433
0.8	0.89	0.92	532	501
0.9	0.97	1.04	580	547
1.0	1.00	1.15	598	564
1.1	0.98	1.27	586	552
1.2	0.92	1.38	550	518
1.3	0.84	1.50	502	473
1.4	0.75	1.61	449	423
1.5	0.66	1.73	395	372
1.6	0.56	1.84	325	316
1.8	0.42	2.07	251	237
2.0	0.32	2.30	191	180
2.2	0.24	2.53	144	136
2.4	0.18	2.76	108	102
2.6	0.13	2.99	78	74
2.8	0.098	3.22	59	56
3.0	0.075	3.45	45	42
3.5	0.036	4.03	21	20
4.0	0.018	4.60	11	10
4.5	0.009	5.18	5	5
5.0	0.004	5.75	2	2
5.5	0.00	6.23	0	0

$\sum_{t=0}^{1.34} q_o = 5752.5$ $\sum_{t=1.84}^{3.45} q_o = 1021$ $\sum_{t=3.45}^{6.23} q_o = 61.5$

RUNOFF(INCHES) = $\sum q \times \Delta t / (645.6 \times DA)$

= $(5752.5 \times .115 + 1021 \times .23 + 61.5 \times .575)/(645.6 \times 1.36)$

= $\underline{1.061''}$ adjust

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NAME OF CLIENT **CORPS OF ENGRS.**

PROJECT **EAST LAKE DAM**

INDIAN FIELDS BRANCH

$$T_P = 1.63 \text{ HRS.} \qquad q_P = 1752 \text{ cfs}$$

T/T_P	q/q_P	TIME (HOURS)	DISCHARGE (CFS)	ADJUSTED q's 1" RUNOFF
0.0	0.00	0.0	0	0
0.1	0.015	0.16	26	26
0.2	0.075	0.33	131	129
0.3	0.16	0.49	280	276
0.4	0.28	0.65	491	483
0.5	0.43	0.82	752	741
0.6	0.60	0.98	1051	1034
0.7	0.77	1.14	1349	1328
0.8	0.89	1.30	1559	1534
0.9	0.97	1.47	1699	1672
1.0	1.00	1.63	1752	(1724)
1.1	0.98	1.79	1717	1690
1.2	0.92	1.96	1612	1586
1.3	0.84	2.12	1472	1449
1.4	0.75	2.28	1314	1293
1.5	0.66	2.45	1156	1138
1.6	0.56	2.61	981	965
1.8	0.42	2.94	736	724
2.0	0.32	3.26	561	552
2.2	0.24	3.59	420	413
2.4	0.18	3.91	315	310
2.6	0.13	4.24	228	224
2.8	0.098	4.57	172	169
3.0	0.075	4.89	131	129
3.5	0.036	5.71	63	62
4.0	0.018	6.52	32	31
4.5	0.009	7.34	16	16
5.0	0.004	8.15	7	7
5.5	0.00	8.97	0	0

$$\overset{2.61}{\underset{0}{\Sigma}} q = 16352.5 \qquad \overset{4.24}{\underset{2.61}{\Sigma}} q = 2988 \qquad \overset{8.97}{\underset{4.24}{\Sigma}} q = 132.5$$

$$\text{RUNOFF (INCHES)} = \Sigma q \times \Delta t / 645.6 \times DA$$

$$= 16352.5 \times .163 + 2988 \times .326 + 132.5 \times .815 / 645.6 \times 5.9$$

$$= 1.016" \quad \text{adjust}$$

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(1")

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PROJECT EAST LAKE DAM

JACKSON RUN BRANCH
.5 HR. UNIT HYDROGRAPH
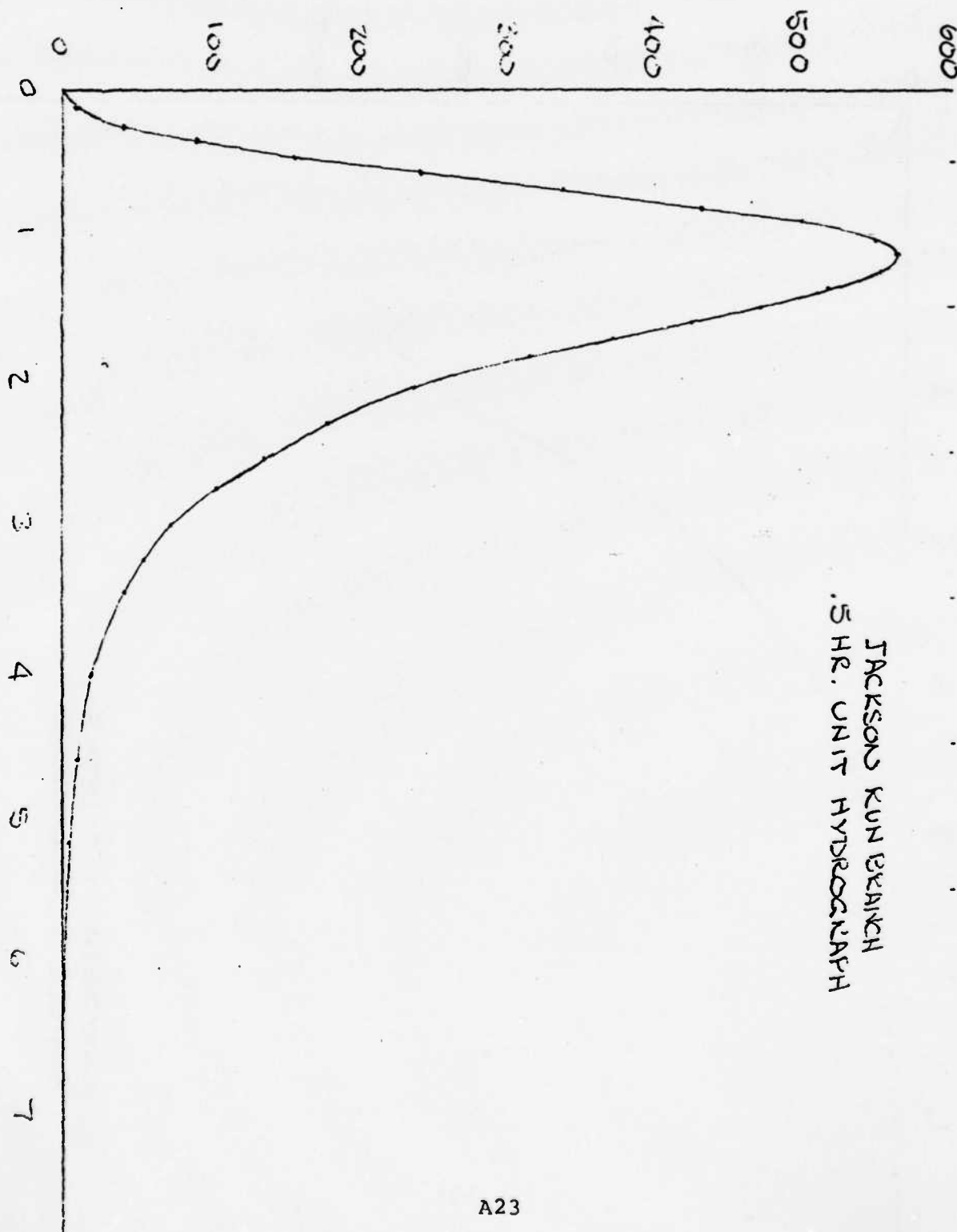
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PROJECT EAST LAKE DAM
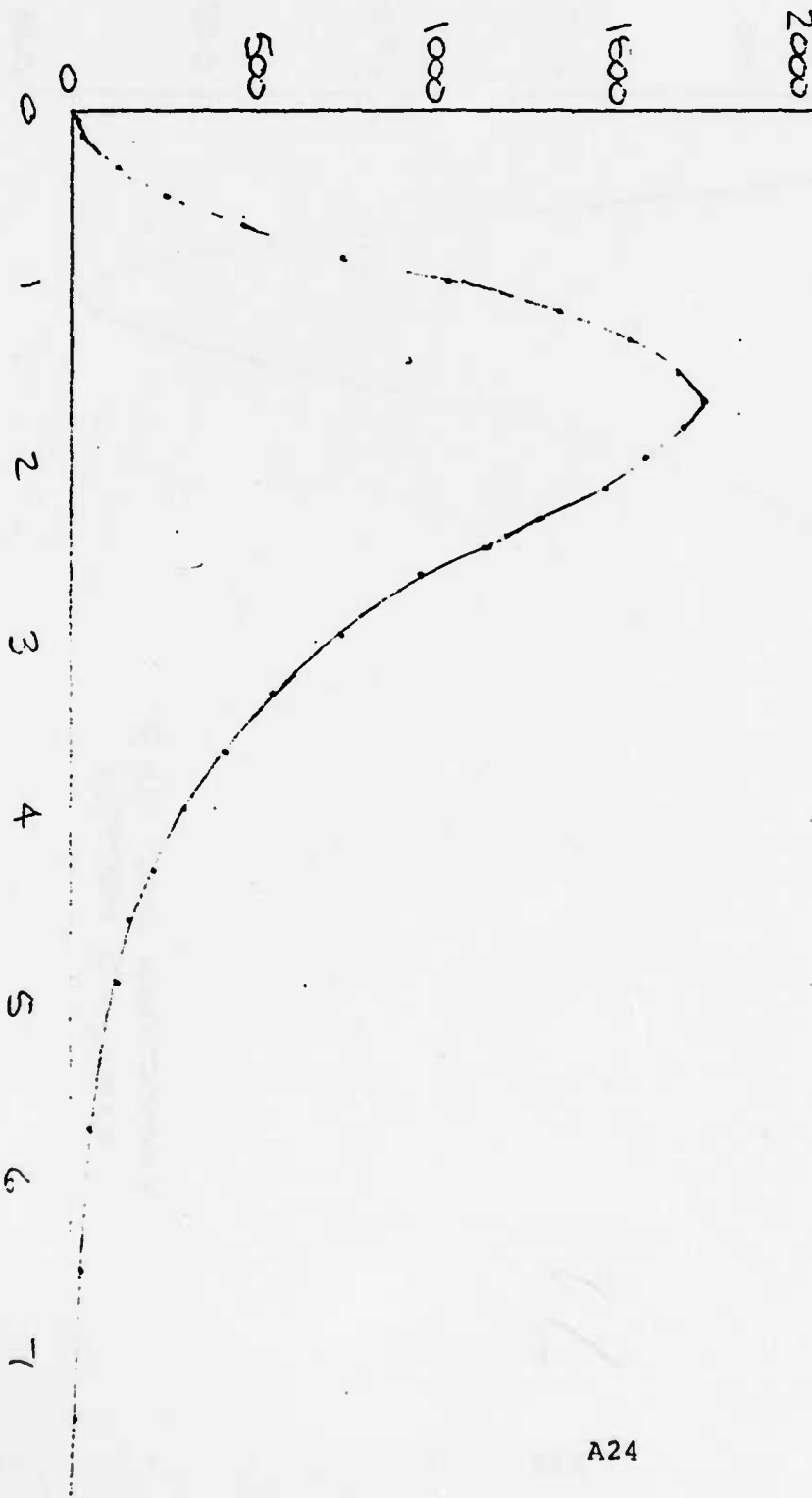
INDIAN FIELDS BRANCH
.5 HOUR UNIT HYDROGRAPH

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NAME OF CLIENT: CORPS OF ENGINEERS

PROJECT: EAST LAKE DAM

— UNIT HYDROGRAPH —
JACKSON RUN BRANCH - ORDINATES FOR 5 MINUTE INCREMENTS

TIME (MIN)	q (CFS)	TIME (MIN)	q (CFS)	TIME (MIN)	qℱ
0	0	185	67	370	1
5	6	190	60	375	1
10	22	195	54	380	0
15	50	200	49		
20	82	205	44		
25	131	210	40		
30	185	215	37		
35	242	220	33		
40	318	225	30		
45	286	230	27		
50	447	235	23		
55	499	240	20		
60	532	245	19		
65	554	250	17		
70	562	255	16		
75	554	260	14		
80	522	265	13		
85	504	270	11		
90	472	275	10		
95	435	280	9		
100	399	285	8		
105	362	290	7		
110	317	295	7		
115	290	300	6		
120	261	305	6		
125	236	310	5		
130	213	315	5		
135	192	320	4		
140	177	325	4		
145	157	330	3		
150	139	335	3		
155	132	340	2		
160	120	345	2		
165	103	350	2		
170	94	355	2		
175	82	360	1		
180	73	365	1		

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NAME OF CLIENT: CORPS OF ENGINEERS

PROJECT: EAST LAKE DAM

— UNIT HYDROGRAPH —
INDIAN FIELDS BRANCH - ORDINATES FOR 5 MINUTE INCREMENT

TIME (MIN)	q (CFS)	TIME (MIN)	q (CFS)	TIME (MIN)	q (CFS)
0	0	185	649	370	44
5	13	190	606	375	41
10	28	195	563	380	38
15	81	200	526	385	35
20	129	205	489	390	32
25	206	210	452	395	30
30	285	215	415	400	29
35	397	220	389	405	27
40	500	225	363	410	26
45	635	230	337	415	24
50	765	235	311	420	22
55	918	240	289	425	21
60	1071	245	266	430	19
65	1224	250	244	435	18
70	1362	255	222	440	16
75	1470	260	208	445	15
80	1560	265	195	450	13
85	1629	270	181	455	12
90	1682	275	167	460	10
95	1709	280	157	465	9
100	1724	285	147	470	7
105	1695	290	137	475	5
110	1663	295	127	480	3
115	1613	300	120	485	1
120	1552	305	114	490	1
125	1480	310	107	495	0
130	1404	315	100		
135	1321	320	94		
140	1244	325	87		
145	1160	330	80		
150	1074	335	73		
155	994	340	67		
160	924	345	60		
165	862	350	57		
170	802	355	54		
175	741	360	51		
180	692	365	48		

NAME OF CLIENT CORPS OF ENGINEERS

PROJECT EAST LAKE DAM

SHEET No. 15 of

DATE 2/23/79

COMP. BY DFC

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STORAGE SURCHARGE (ACRE-FEET)

STAGE (FT ABOVE MSL)

(ELEV. 20.0)

A27
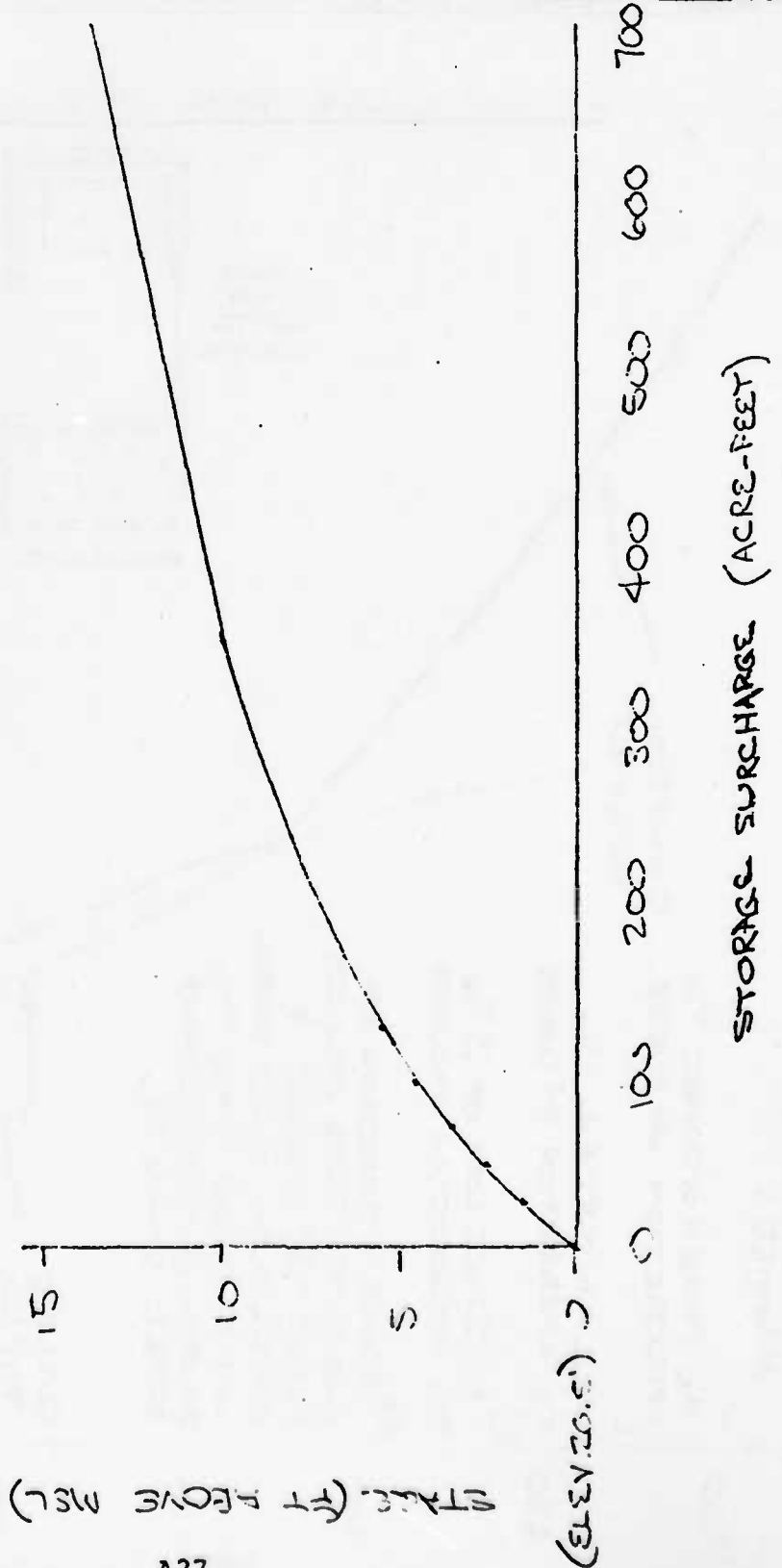
RATING CURVE FOR EAST OUTLET

WEIR OPENINGS 2'x4' (ONE EACH SIDE)

TWO 48" GATED OPENINGS

TWO 42" CMP LINERS

TWO 48" C.I.P.

NOT TO SCALE

SIZES ARE APPROXIMATE

CONTROL CURVE

48" GATED OPENINGS & OVERFLOW WEIRS

OUTLET PIPES

ASSUMED:

① d_c THROUGH OPENINGS TO INTERSECTION OF CURVES.

② W.S.E. = INVERT + d_c + H_v TO INTERSECTION OF CURVES.

③ ENTRANCE LOSS OF .3 H_v TO INTERSECTION OF CURVES

④ BEYOND INTERSECTION OF CURVES THE GATED OPENINGS FLOOD THE CHAMBER & ORIFICE FLOW WOULD CONTROL OUTLET FLOW SINCE THE SLOPE IS HYDRAULICALLY STEEP (S=4.6%).

300

250

200

150

100

50

0

32 30 28 26 24 22 20

A28

Division of O'Brien & Gere Engineers, Inc. SHEET NO. 11 OF ____
PHILADELPHIA, PA
DATE 2/28/78
NAME OF CLIENT CORPS OF ENGINEERS
COMP. BY DBC
PROJECT EAST LAKE DAM
CHECKED BY LRW

THE WEST OUTLET CHANNEL HAS A VERY MINOR CAPACITY AND CAN BE IGNORED IN CONJUNCTION WITH THE SPILLWAY DESIGN FLOOD. SHEET FLOW THROUGH THIS AREA IS CONSIDERED.

OVERFLOW

② STONE WALL ALONG COMMERCE STREET AND EXTENDING ACROSS POLE ESTATE.

$$L = 830' \quad ELEV \approx 27.5' (MSL) \quad C = 3.0$$

③ AREA TO NORTH SIDE OF POLE ESTATE INCLUDING THE WEST OUTLET (BEYOND THE WALL).

$$L \approx 100' \quad ELEV \approx 26.0' MSL \quad C \approx 2.6$$

A29

JUSTIN & COURTNEY, INC.
Division of O'Brien & Gere Engineers, Inc.
PHILADELPHIA, PA

SHEET NO. 18 OF

DATE 2/28/78

COMP. BY DBC

CHECKED BY LRW
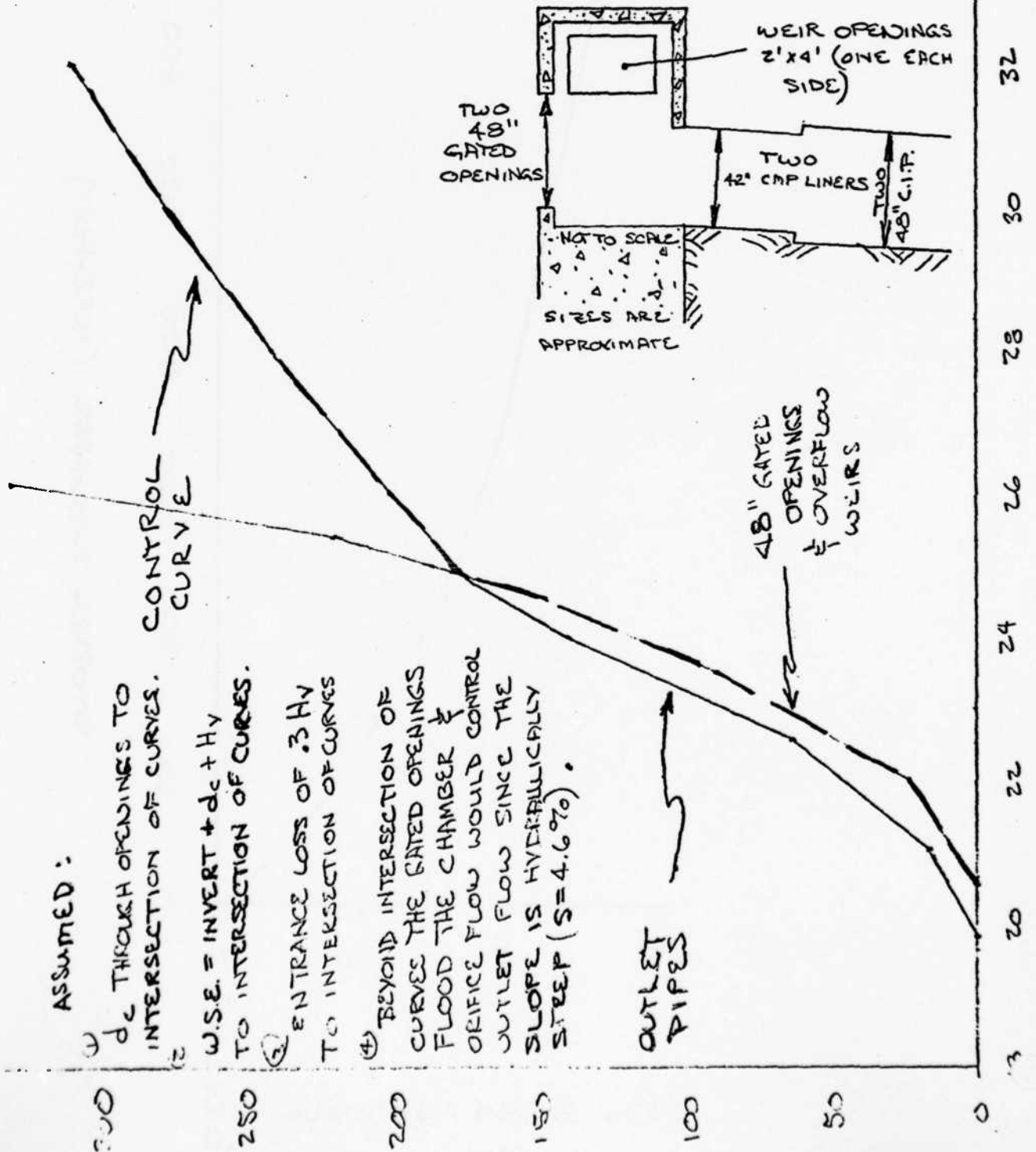
NAME OF CLIENT CORPS OF ENGINEERS

PROJECT EAST LAKE DAM

ELEVATION (FEET ABOVE MSL)
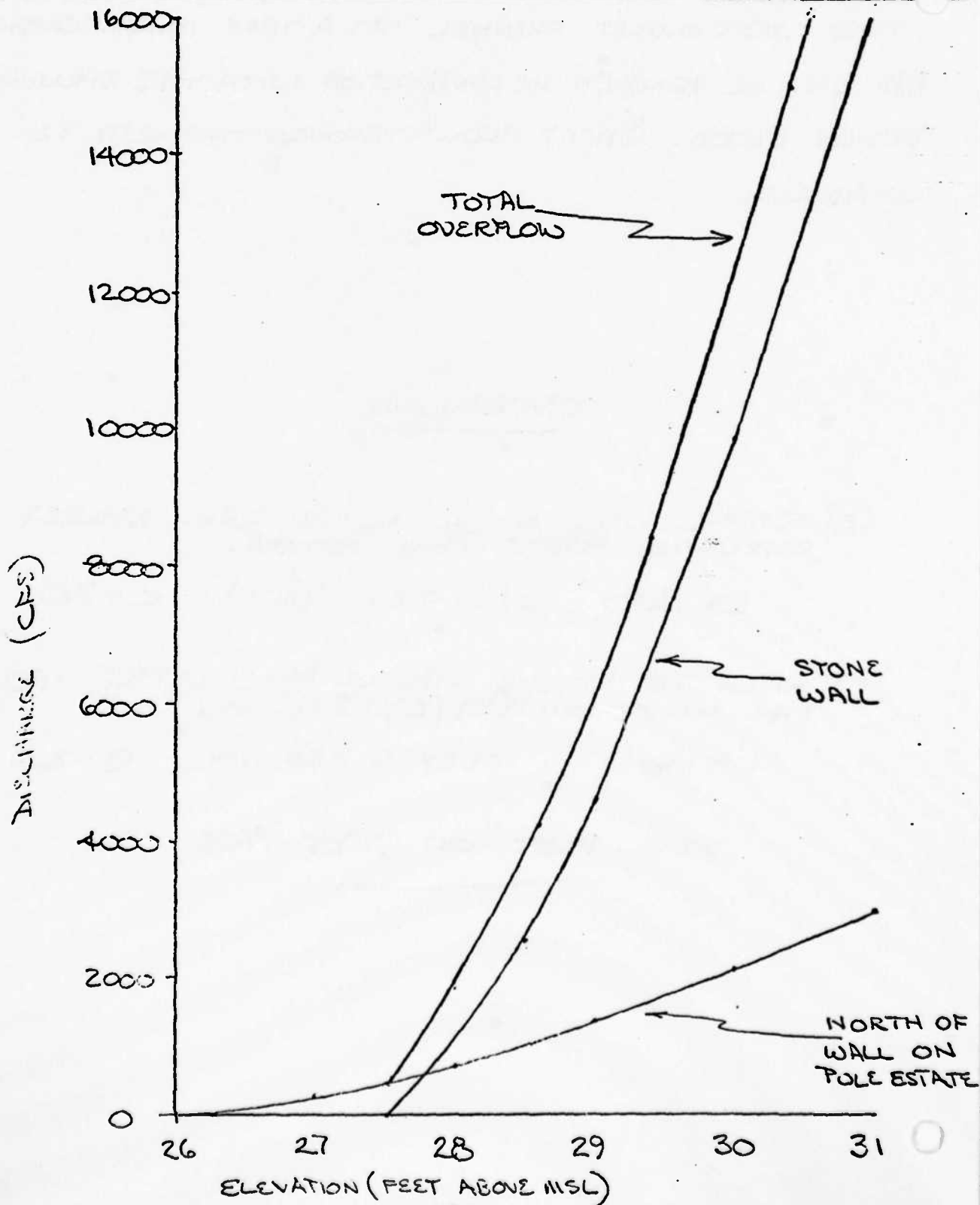
A30

Division of O'Brien & Gere Engineers, Inc.   SHEET NO. 14 OF _____
PHILADELPHIA, PA

NAME OF CLIENT ___ CORPS OF ELGRS ___

PROJECT ___ EAST LAKE DAM ___

DATE 2/28/78

COMP. BY DBC

CHECKED BY LRW

TOTAL STAGE - DISCHARGE RELATION

ELEV (MSL)	STAGE (FT)	EAST OUTLET	TOTAL OVERFLOW	TOTAL
20.5	0	0	0	0
22.0	1.5	20	0	20
24.0	3.5	120	0	120
26.0	5.5	200	0	200
27.0	6.5	220	260	480
28.0	7.5	240	2000	2240
29.0	8.5	260	6000	6260
29.5	9.0	270	9000	9270
30.0	9.5	280	12400	12680
30.5	10.0	285	16000	16285

JUSTIN & COURTNEY, INC.
Division of O'Brien & Gere Engineers, Inc. SHEET NO. _20_ OF _____

PHILADELPHIA, PA

DATE 2/28/78

NAME OF CLIENT _CORPS OF ENGRS._

COMP. BY _DBC_

PROJECT _EAST LAKE DAM_

CHECKED BY _LRW_

STORAGE-DISCHARGE RELATION

SURCHARGE STAGE	SURCHARGE STORAGE	DISCHARGE
0	0	0
1.5	25	20
3.5	68	120
5.5	125	200
6.5	170	480
7.5	210	2240
8.5	260	6260
9.0	290	9270
9.5	320	12680
10.0	350	16285

A32

HEC-1 COMPUTATIONS

NATIONAL DAM SAFETY PROGRAM
EAST LAKE DAM
ONE HALF PROBABLE MAXIMUM FLOOD COMPUTATION

JOB SPECIFICATION

NQ	NHR	NMIN	IDAY	IHR	IMIN	METRC	IPLT	IPRT	NSTAN
150	0	5	1	0	0	0	0	2	2	0

JOPER   NHT
3   0

*********   *********   *********   *********

SUB-AREA RUNOFF COMPUTATION

HYDROGRAPH COMPUTATION FOR THE JACKSON RUN BRANCH.

	ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPPT	INANF
	1	0	0	0	0	0	1

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
0	-1	1.36	0.00	0.00	0.00	0.000	0	0	0

PRECIP DATA

	NP	STORM	DAJ	DAK
	61	0.00	0.00	0.00

PRECIP PATTERN

	0.00	0.00	.39	0.00	0.00
	0.00	0.00	0.00	0.00	.45
	0.00	.55	0.00	0.00	0.00
	0.00	0.07	3.15	0.00	0.70
	0.00	0.40	0.00	0.00	0.00

LOSS DATA

STRKR	DLTKR	RTIOL	ERAIN	STRKS	DLTKS	RTIOK	STRTL	STPTL	CNSTL	ALSMX	RTIMP
0.00	0.00	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.00	0.00

GIVEN UNIT GRAPH, NUHGQ= 77

.15	6.	22.	50.	43.	131.	145.	242.	318.	386.
0.00	493.	512.	554.	563.	554.	512.	504.	477.	475.
0.00	352.	317.	290.	251.	236.	217.	192.	177.	158.
1.90	135.	120.	103.	94.	83.	73.	67.	60.	54.
0.00	44.	40.	37.	33.	30.	27.	23.	20.	19.
0.00	17.	14.	13.	11.	10.	9.	7.	7.	7.
.40	6.	5.	4.	4.	3.	3.	2.	2.	2.
	2.	1.	1.	1.	0.				

UNIT GRAPH TOTALS   10526. CFS OR 1.00 INCHES OVER THE AREA

RECESSION DATA

STRTQ= 0.00   QRCSN= 0.00   RTIOR= 1.00

1	0	0.15	0.00	0.00	3.
1	0	0.20	0.00	0.00	5.
1	0	0.25	0.00	0.00	12.
1	0	0.30	.30	.30	20.
1	0	0.35	0.00	0.00	28.
1	0	0.40	0.00	0.00	38.
1	0	0.45	0.00	0.00	54.
1	0	0.50	0.00	0.00	77.
1	0	0.55	0.00	0.00	92.
1	0	0.60	.35	.35	114.
1	1	1.05	0.00	0.00	135.
1	1	1.10	0.00	0.00	158.
1	1	1.15	0.00	0.00	183.
1	1	1.20	0.00	0.00	216.
1	1	1.25	0.00	0.00	243.
1	1	1.30	0.00	0.00	271.
1	1	1.35	.45	.45	295.
1	1	1.40	0.00	0.00	319.
1	1	1.45	0.00	0.00	350.
1	1	1.50	0.00	0.00	378.
1	1	1.55	0.00	0.00	401.
1	1	1.60	0.00	0.00	428.
1	2	2.05	.55	.55	451.
1	2	2.10	0.00	0.00	472.
1	2	2.15	0.00	0.00	504.
1	2	2.20	0.00	0.00	533.
1	2	2.25	0.00	0.00	555.
1	2	2.30	0.00	0.00	584.
1	2	2.35	1.90	1.90	606.
1	2	2.43	0.00	0.00	637.
1	2	2.50	0.00	0.00	692.
1	2	2.55	0.00	0.00	756.
1	2	2.60	0.00	0.00	821.
1	3	3.05	0.00	0.00	911.
1	3	3.10	0.00	0.00	1001.
1	3	3.15	3.15	3.15	1112.
1	3	3.20	0.00	0.00	1202.
1	3	3.25	0.00	0.00	1465.
1	3	3.30	0.00	0.00	1644.
1	3	3.35	.90	.90	1855.
1	3	3.40	0.00	0.00	2044.
1	3	3.45	0.00	0.00	2234.
1	3	3.50	0.00	0.00	2471.
1	3	3.55	0.00	0.00	2656.
1	3	3.60	0.00	0.00	2799.
1	4	4.05	.70	.70	2924.
1	4	4.10	0.00	0.00	2988.
1	4	4.15	0.00	0.00	3021.
1	4	4.20	0.00	0.00	3033.
1	4	4.25	0.00	0.00	2998.
1	4	4.30	.40	.40	2906.
1	4	4.35	0.00	0.00	2828.
1	4	4.40	0.00	0.00	2725.
1	4	4.45	0.00	0.00	2617.
1	4	4.50	0.00	0.00	2511.
1	4	4.55	.40	.40	2391.
1	4	4.60	0.00	0.00	2248.
1	5	5.05	0.00	0.00	2147.
1	5	5.10	0.00	0.00	2028.
1	5	5.15	0.00	0.00	1939.
1	5	5.20	0.00	0.00	1845.
1	5	5.25	0.00	0.00	1740.
1	5	5.30	0.00	0.00	1653.

1	5	35	0.00	0.00	0.00	1466.
1	5	40	0.00	0.00	0.00	1416.
1	5	45	0.00	0.00	0.00	1346.
1	5	50	0.00	0.00	0.00	1257.
1	5	55	0.00	0.00	0.00	1187.
1	5	60	0.00	0.00	0.00	1114.
1	6	5	0.00	0.00	0.00	1035.
1	6	10	0.00	0.00	0.00	980.
1	6	15	0.00	0.00	0.00	909.
1	6	20	0.00	0.00	0.00	835.
1	6	25	0.00	0.00	0.00	767.
1	6	30	0.00	0.00	0.00	697.
1	6	35	0.00	0.00	0.00	631.
1	6	40	0.00	0.00	0.00	584.
1	6	45	0.00	0.00	0.00	527.
1	6	50	0.00	0.00	0.00	474.
1	6	55	0.00	0.00	0.00	426.
1	6	60	0.00	0.00	0.00	380.
1	7	10	0.00	0.00	0.00	336.
1	7	15	0.00	0.00	0.00	311.
1	7	20	0.00	0.00	0.00	280.
1	7	25	0.00	0.00	0.00	252.
1	7	30	0.00	0.00	0.00	228.
1	7	35	0.00	0.00	0.00	205.
1	7	40	0.00	0.00	0.00	179.
1	7	45	0.00	0.00	0.00	166.
1	7	50	0.00	0.00	0.00	150.
1	7	55	0.00	0.00	0.00	133.
1	7	60	0.00	0.00	0.00	119.
1	8	5	0.00	0.00	0.00	107.
1	8	10	0.00	0.00	0.00	94.
1	8	15	0.00	0.00	0.00	85.
1	8	20	0.00	0.00	0.00	77.
1	8	25	0.00	0.00	0.00	69.
1	8	30	0.00	0.00	0.00	63.
1	8	35	0.00	0.00	0.00	55.
1	8	40	0.00	0.00	0.00	48.
1	8	45	0.00	0.00	0.00	45.
1	8	50	0.00	0.00	0.00	39.
1	8	55	0.00	0.00	0.00	36.
1	8	60	0.00	0.00	0.00	31.
1	9	5	0.00	0.00	0.00	28.
1	9	10	0.00	0.00	0.00	22.
1	9	15	0.00	0.00	0.00	21.
1	9	20	0.00	0.00	0.00	19.
1	9	25	0.00	0.00	0.00	17.
1	9	30	0.00	0.00	0.00	13.
1	9	35	0.00	0.00	0.00	12.
1	9	40	0.00	0.00	0.00	10.
1	9	45	0.00	0.00	0.00	9.
1	9	50	0.00	0.00	0.00	8.
1	9	55	0.00	0.00	0.00	7.
1	9	60	0.00	0.00	0.00	6.
1	10	5	0.00	0.00	0.00	5.
1	10	10	0.00	0.00	0.00	4.
1	10	15	0.00	0.00	0.00	4.
1	10	20	0.00	0.00	0.00	4.
1	10	25	0.00	0.00	0.00	2.
1	10	30	0.00	0.00	0.00	2.
1	10	35	0.00	0.00	0.00	2.
1	10	45	0.00	0.00	0.00	1.
1	10	50	0.00	0.00	0.00	1.

OVN

SUB-AREA RUNOFF COMPUTATION

HYDROGRAPH COMPUTATION FOR THE INDIAN FIELDS BRANCH.

	ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME		
	2	0	0	0	0	0	1		

	IHYDG	IUHG	TAREA	SNAP	HYDROGRAPH DATA TRSDA	TRSPC	RATIO	ISNOW	TSAME	LOCAL
	0	-1	5.90	0.00	0.00	0.00	0.000	0	0	0

PRECIP DATA

	NP	STORM	DAJ	DAK
	61	0.00	0.00	0.00

PRECIP PATTERN

.05	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.00	.20	0.00	0.00	.15	0.00	0.00	.35
0.00	0.00	.50	0.00	0.00	0.00	0.00	0.00
1.70	.90	0.00	0.00	3.00	0.00	0.00	.70
0.00	0.00	.40	0.00	0.00	0.00	0.00	0.00
.40

LOSS DATA

STRKR	DLTKR	RTIOL	ERAIN	STPKS	RTIOK	STRTL	GNSTL	ALSNK	RTIMP
0.00	0.00	1.00			1.00		0.00	0.00	0.00

GIVEN UNIT GRAPH, NUMQG= 100

0.	13.	28.	81.	129.	206.	245.	397.	500.	675.
765.	918.	1071.	1224.	1362.	1470.	1560.	1629.	1682.	1709.
1724.	1695.	1663.	1613.	1557.	1480.	1404.	1371.	1344.	1168.
1084.	996.	924.	863.	802.	741.	692.	649.	606.	567.
526.	489.	452.	415.	389.	363.	337.	311.	289.	266.
244.	222.	208.	195.	181.	167.	157.	147.	137.	127.
120.	114.	107.	100.	94.	87.	80.	73.	67.	60.
57.	56.	51.	48.	44.	41.	38.	35.	32.	30.
23.	27.	26.	24.	22.	21.	19.	18.	16.	15.
13.	12.	10.	9.	7.	5.	3.	1.	1.	0.

UNIT GRAPH TOTALS 45678. CFS OR 1.00 INCHES OVER THE AREA

RECESSION DATA

STRTQ= 0.00   QRCSN= 0.00   RTIOR= 1.00

END-OF-PERIOD FLOW

TIME	RAIN	EXCS	COMP Q
1 0 5	.05	.05	0.
1 0 10	0.00	0.00	1.
1 0 15	0.00	0.00	1.
1 0 20	0.00	0.00	4.
1 0 25	0.00	0.00	6.
1 0 30	0.00	0.00	10.
1 0 35	.15	.15	14.
1 0 40	0.00	0.00	22.
1 0 45	0.00	0.00	32.
1 0 50	0.00	0.00	44.
1 0 55	0.00	0.00	58.

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96.	.20	.20	1	5	1
123.	0.00	0.00	1	10	1
149.	0.02	0.00	1	15	1
185.	0.00	0.00	1	20	1
219.	0.00	0.00	1	25	1
260.	.35	.35	1	30	1
302.	0.00	0.00	1	35	1
353.	0.00	0.00	1	40	1
400.	0.00	0.00	1	45	1
461.	0.00	0.00	1	50	1
515.	.50	.50	1	55	1
581.	0.00	0.00	1	60	1
644.	0.00	0.00	2	5	1
721.	0.00	0.00	2	10	1
790.	0.00	0.00	2	15	1
877.	1.70	1.70	2	20	1
956.	0.00	0.00	2	25	1
1050.	0.00	0.00	2	30	1
1141.	0.00	0.00	2	35	1
1263.	0.00	0.00	2	40	1
1376.	3.00	3.00	2	45	1
1550.	0.00	0.00	2	50	1
1707.	0.00	0.00	2	55	1
1916.	0.00	0.00	2	60	1
2116.	0.00	0.00	3	5	1
2402.	.90	.90	3	10	1
2668.	0.00	0.00	3	15	1
3073.	0.00	0.00	3	20	1
3445.	0.00	0.00	3	25	1
3927.	0.00	0.00	3	30	1
4403.	.70	.70	3	35	1
4973.	0.00	0.00	3	40	1
5489.	0.00	0.00	3	45	1
6062.	0.00	0.00	3	50	1
6586.	0.00	0.00	3	55	1
7161.	0.00	0.00	3	60	1
7706.	.40	.40	4	5	1
8237.	0.00	0.00	4	10	1
8704.	0.00	0.00	4	15	1
9060.	0.00	0.00	4	20	1
9355.	0.00	0.00	4	25	1
9597.	.40	.40	4	30	1
9775.	0.00	0.00	4	35	1
9883.	0.00	0.00	4	40	1
9942.	0.00	0.00	4	45	1
9872.	0.00	0.00	4	50	1
9765.	.40	.40	4	55	1
9671.	0.00	0.00	4	60	1
9742.	0.00	0.00	5	5	1
9016.	0.00	0.00	5	10	1
8751.	0.00	0.00	5	15	1
8600.	0.00	0.00	5	20	1
8221.	0.00	0.00	5	25	1
7930.	0.00	0.00	5	30	1
7617.	0.00	0.00	5	35	1
7348.	0.00	0.00	5	45	1
7070.	0.00	0.00	5	50	1
6600.	0.00	0.00			1

A38

********** ********** ********** ********** ********** ********** **********

COMBINE HYDROGRAPHS

COMBINED INFLOW HYDROGRAPH INTO EAST LAKE RESERVOIR

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
3	2	0	0	0	0	1
3						

SUM OF 2 HYDROGRAPHS AT   3

0.	2.	5.	12.	19.	30.	42.	60.	53.	117.
150.	191.	232.	281.	336.	401.	462.	532.	597.	672.
750.	813.	916.	1009.	1024.	1193.	1294.	1410.	1511.	1674.
1747.	1899.	2068.	2306.	2528.	2826.	3117.	3513.	3950.	4577.
5083.	5782.	6451.	7211.	7951.	8718.	9345.	10085.	10593.	11258.
11741.	12205.	12260.	12425.	12500.	12496.	12453.	12264.	12036.	11887.
11499.	11181.	10861.	10491.	10142.	9784.	9397.	9033.	8594.	8727.
7987.	7632.	7249.	6963.	6648.	6302.	5941.	5643.	5311.	4943.
4590.	4386.	4108.	3827.	3560.	3308.	3075.	2845.	2654.	2465.
2281.	2111.	1365.	1821.	1694.	1568.	1456.	1355.	1260.	1154.
1088.	1003.	933.	863.	797.	734.	645.	634.	590.	560.
503.	470.	437.	405.	373.	348.	325.	301.	281.	250.
238.	223.	205.	189.	173.	158.	140.	128.	115.	104.
92.	82.	70.	60.	55.	48.	43.	39.	33.	30.
27.	24.	22.	19.	16.	14.	12.		9.	8.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	12500.	6232.	3192.	3192.	478750.
INCHES		8.05	8.52	8.52	8.52
AC-FT		3117.	3299.	3299.	3299.

HYDROGRAPH ROUTING

ROUTED HYDROGRAPH

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
4	1	0	0	0	0	1

ROUTING DATA

QLOSS	CLOSS	AVG	IRFS	ISAME
0.0	0.000	0.00	1	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA
0	0	0.000	0.000	0.000	0.000	-1.

| 25. | 69. | 125. | 170. | 210. | 260. | 230. | 320. | 350. |
| 20. | 120. | 200. | 400. | 2240. | 6260. | 9270. | 12680. | 16785. |

STORAGE= 0.
OUTFLOW= 0.

TIME			EOP STOR	AVG IN	EOP OUT
1	0	5	0.	0.	0.
1	0	10	0.	1.	0.
1	0	15	0.	3.	0.
1	0	20	0.	8.	0.
1	0	25	0.	15.	0.
1	0	30	0.	24.	0.
1	0	35	1.	35.	0.
1	0	40	1.	51.	1.
1	0	45	1.	72.	1.
1	0	50	2.	100.	2.
1	0	55	3.	133.	2.
1	0	60	4.	170.	3.
1	1	05	6.	211.	4.
1	1	10	7.	256.	6.
1	1	15	9.	309.	8.
1	1	20	12.	369.	10.
1	1	25	15.	431.	12.
1	1	30	18.	497.	14.
1	1	35	22.	564.	16.
1	1	40	26.	634.	32.
1	1	45	31.	711.	34.
1	1	50	36.	794.	46.
1	1	55	42.	877.	59.
1	1	60	48.	963.	73.
1	2	05	55.	1052.	89.
1	2	10	29.	1153.	109.
1	2	15	07.	1243.	122.
1	2	20	78.	1352.	134.
1	2	25	87.	1460.	147.
1	2	30	97.	1572.	160.
1	2	35	107.	1690.	175.
1	2	40	119.	1825.	191.
1	2	45	131.	1963.	236.
1	2	50	144.	2187.	318.
1	2	55	154.	2417.	406.
1	2	60	173.	2677.	614.
1	3	05	187.	2971.	1234.
1	3	10	200.	3315.	1782.
1	3	15	211.	3732.	2131.

1 3 25	230.	4413.	3177.
1 3 30	239.	5435.	4553.
1 3 35	247.	6116.	5231.
1 3 40	256.	6831.	5725.
1 3 45	265.	7586.	6716.
1 3 50	273.	8339.	7550.
1 3 55	281.	9052.	8321.
1 3 60	288.	9735.	9047.
1 4 05	294.	10389.	9781.
1 4 10	300.	10975.	10453.
1 4 15	306.	11499.	11042.
1 4 20	310.	11900.	11524.
1 4 25	313.	12157.	11891.
1 4 30	315.	12343.	12141.
1 4 35	317.	12462.	12322.
1 4 40	318.	12498.	12421.
1 4 45	318.	12474.	12451.
1 4 50	318.	12358.	12399.
1 4 55	316.	12149.	12258.
1 4 60	315.	11918.	12067.
1 5 05	313.	11651.	11833.
1 5 10	310.	11340.	11556.
1 5 15	307.	11021.	11255.
1 5 20	305.	10676.	10929.
1 5 25	302.	10317.	10585.
1 5 30	298.	9963.	10235.
1 5 35	295.	9590.	9872.
1 5 40	292.	9215.	9502.
1 5 45	289.	8863.	9154.
1 5 50	286.	8511.	8824.
1 5 55	282.	8157.	8481.
1 5 60	279.	7800.	9136.
1 6 05	275.	7460.	7789.
1 6 10	272.	7126.	7469.
1 6 15	269.	6806.	7119.
1 6 20	265.	6475.	6788.
1 6 25	262.	6141.	6456.
1 6 30	259.	5812.	6146.
1 6 35	255.	5477.	5856.
1 6 40	251.	5147.	5549.
1 6 45	247.	4836.	5240.
1 6 50	244.	4538.	4935.
1 6 55	240.	4247.	4637.
1 7 05	236.	3967.	4346.
1 7 10	233.	3694.	4063.
1 7 15	229.	3434.	3791.
1 7 20	226.	3192.	3531.
1 7 25	223.	2961.	3284.
1 7 30	220.	2750.	3052.
1 7 35	217.	2559.	2838.
1 7 40	215.	2373.	2637.
1 7 45	212.	2196.	2446.
1 7 50	210.	2038.	2269.
1 7 55	208.	1893.	2158.
1 7 60	206.	1758.	2053.
1 8 05	203.	1631.	1742.
1 8 10	201.	1512.	1829.
1 8 15	198.	1405.	1717.
1 8 20	196.		1609.
1 8 25	193.	1212.	1505.
1 8 30	191.	1114.	1406.

Time			Q1	Q2	Q3
1	9	5	177.	640.	640.
1	9	10	175.	570.	727.
1	9	15	174.	529.	675.
1	9	20	173.	487.	626.
1	9	25	172.	455.	581.
1	9	30	171.	421.	539.
1	9	35	170.	387.	499.
1	9	40	169.	360.	477.
1	9	45	168.	336.	471.
1	9	50	166.	313.	465.
1	9	55	165.	291.	457.
1	9	60	164.	271.	450.
1	10	5	162.	251.	441.
1	10	10	161.	231.	432.
1	10	15	159.	214.	423.
1	10	20	158.	197.	414.
1	10	25	156.	181.	406.
1	10	30	155.	165.	394.
1	10	35	153.	149.	384.
1	10	40	151.	136.	373.
1	10	45	149.	121.	363.
1	10	50	148.	109.	352.
1	10	55	146.	98.	341.
1	10	60	144.	87.	331.
1	11	5	143.	76.	320.
1	11	10	141.	65.	309.
1	11	15	139.	58.	299.
1	11	20	138.	52.	288.
1	11	25	136.	46.	278.
1	11	30	134.	41.	268.
1	11	35	133.	36.	258.
1	11	40	131.	32.	249.
1	11	45	130.	28.	240.
1	11	50	129.	26.	231.
1	11	55	127.	23.	222.
1	11	60	126.	20.	214.
1	12	5	125.	18.	205.
1	12	10	123.	15.	199.
1	12	15	123.	13.	198.
1	12	20	122.	12.	196.
1	12	25	121.	10.	194.
1	12	30	119.	9.	192.

SUM   461496.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	124471.	6149.	3077.	3077.	461495.
INCHES		7.58	8.21	8.21	8.21
AC-FT		7051.	3190.	3187.	3180.

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INFLOW(I), OUTFLOW(O) AND OBSERVED FLOW(C)

RECOMMENDED GUIDELINES

FOR SAFETY INSPECTION OF DAMS

CHAPTER 4

CHAPTER 4 - PHASE II INVESTIGATION

4.1. Purpose. The Phase II investigation will be supplementary to Phase I and should be conducted when the results of the Phase I investigation indicate the need for additional in-depth studies, investigations or analyses.

4.2. Scope. The Phase II investigation should include all additional studies, investigations and analyses necessary to evaluate the safety of the dam. Included, as required, will be additional visual inspections, measurements, foundation exploration and testing, materials testing, hydraulic and hydrologic analysis and structural stability analyses.

4.3. Hydraulic and Hydrologic Analysis. Hydraulic and hydrologic capabilities should be determined using the following criteria and procedures. Depending on the project characteristics, either the spillway design flood peak inflow or the spillway design flood hydrograph should be the basis for determining the maximum water surface elevation and maximum outflow. If the operation or failure of upstream water control projects would have significant impact on peak flow or hydrograph analyses, the impact should be assessed.

4.3.1. Maximum Water Surface Based on SDF Peak Inflow. When the total project discharge capability at maximum pool exceeds the peak inflow of the recommended SDF, and operational constraints would not prevent such a release at controlled projects, a reservoir routing is not required. The maximum discharge should be assumed equal to the peak inflow of the spillway design flood. Flood volume is not controlling in this situation and surcharge storage is either absent or is significant only to the extent that it provides the head necessary to develop the release capability required.

4.3.1.1. Peak for 100-Year Flood. When the 100-year flood is applicable under the provisions of Table 3 and data are available, the spillway design flood peak inflow may be determined by use of "A Uniform Technique for Determining Flood Frequencies," Water Resources Council (WRC), Hydrology Committee, Bulletin 15, December 1967. Flow frequency information from regional analysis is generally preferred over single station results when available and appropriate. Rainfall-runoff techniques may be necessary when there are inadequate runoff data available to make a reasonable estimate of flow frequency.

4.3.1.2. Peak for PMF or Fraction Thereof. When either the Probable Maximum Flood peak or a fraction thereof is applicable under the provisions of Table 3, the unit hydrograph - infiltration loss technique is generally the most expeditious method of computing the spillway design flood peak for most projects. This technique is discussed in the following paragraph.

4.3.2. Maximum Water Surface Based on SDF Hydrograph. Both peak and
volume are required in this analysis. Where surcharge storage is signifi-
cant, or where there is insufficient discharge capability at maximum pool
to pass the peak inflow of the SDF, considering all possible operational
constraints, a flood hydrograph is required. When there are upstream
hazard areas that would be imperiled by fast rising reservoirs levels, SDF
hydrographs should be routed to ascertain available time for warning
and escape. Determination of probable maximum precipitation or 100-year
precipitation, whichever is applicable, and unit hydrographs or runoff
models will be required, followed by the determination of the PMF or 100-
year flood. Conservative loss rates (significantly reduced by antecedent
rainfall conditions where appropriate) should be estimated for computing
the rainfall excess to be utilized with unit hydrographs. Rainfall values
are usually arranged with gradually ascending and descending rates with
the maximum rate late in the storm. When applicable, conservatively high
snowmelt runoff rates and appropriate releases from upstream projects
should be assumed. The PMP may be obtained from National Weather
Service (NWS) publications such as Hydrometeorological Report (HMR) 33.
Special NWS publications for particular areas should be used when avail-
able. Rainfall for the 100-year frequency flood can be obtained from the
NWS publication "Rainfall Frequency Atlas of the United States," Technical
Paper No. 40; Atlas 2, "Precipitation Frequency Atlas of Western United
States;" or other NWS publications. The maximum water surface elevation
and spillway design flood outflow are then determined by routing the inflow
hydrograph through the reservoir surcharge storage, assuming a starting
water surface at the bottom of surcharge storage, or lower when appropriate.
For projects where the bottom of surcharge space is not distinct, or the
flood control storage space (exclusive of surcharge) is appreciable, it
may be appropriate to select starting water surface elevations below the
top of the flood control storage for routings. Conservatively high
starting levels should be estimated on the basis of hydrometeorological
conditions reasonably characteristic for the region and flood release
capability of the project. Necessary adjustment of reservoir storage
capacity due to existing or future sediment or other encroachment may be
approximated when accurate determination of deposition is not practicable.

4.3.3. Acceptable Procedures. Techniques for performing hydraulic and
hydrologic analyses are generally available from publications prepared by
Federal agencies involved in water resources development or textbooks
written by the academic community. Some of these procedures are rather
sophisticated and require expensive computational equipment and large data
banks. While results of such procedures are generally more reliable than
simplified methods, their use is generally not warranted in studies con-
nected with this program unless they can be performed quickly and inexpen-
sively. There may be situations where the more complex techniques have
to be employed to obtain reliable results; however, these cases will be
exceptions rather than the rule. Whenever the acceptability of procedures
is in question, the advice of competent experts should be sought. Such
expertise is generally available in the Corps of Engineers, Bureau of

Reclamation and Soil Conservation Service. Many other agencies, educational facilities and private consultants can also provide expert advice. Regardless of where such expertise is based, the qualification of those individuals offering to provide it should be carefully examined and evaluated.

4.3.4. Freeboard Allowances. Guidelines on specific minimum freeboard allowances are not considered appropriate because of the many factors involved in such determinations. The investigator will have to assess the critical parameters for each project and develop its minimum requirement. Many projects are reasonably safe without freeboard allowance because they are designed for overtopping, or other factors minimize possible overtopping. Conversely, freeboard allowances of several feet may be necessary to provide a safe condition. Parameters that should be considered include the duration of high water levels in the reservoir during the design flood; the effective wind fetch and reservoir depth available to support wave generation; the probability of high wind speed occurring from a critical direction; the potential wave runup on the dam based on roughness and slope; and the ability of the dam to resist erosion from overtopping waves.

4.4. Stability Investigations. The Phase II stability investigations should be compatible with the guidelines of this paragraph.

4.4.1. Foundation and Material Investigations. The scope of the foundation and materials investigation should be limited to obtaining the information required to analyze the structural stability and to investigate any suspected condition which would adversely affect the safety of the dam. Such investigations may include borings to obtain concrete, embankment, soil foundation, and bedrock samples; testing specimens from these samples to determine the strength and elastic parameters of the materials, including the soft seams, joints, fault gouge and expansive clays or other critical materials in the foundation; determining the character of the bedrock including joints, bedding planes, fractures, faults, voids and caverns, and other geological irregularities; and installing instruments for determining movements, strains, suspected excessive internal seepage pressures, seepage gradients and uplift forces. Special investigations may be necessary where suspect rock types such as limestone, gypsum, salt, basalt, claystone, shales or others are involved in foundations or abutments in order to determine the extent of cavities, piping or other deficiencies in the rock foundation. A concrete core drilling program should be undertaken only when the existence of significant structural cracks is suspected or the general qualitative condition of the concrete is in doubt. The tests of materials will be necessary only where such data are lacking or are outdated.

4.4.2. Stability Assessment. Stability assessments should utilize in situ properties of the structure and its foundation and pertinent geologic

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information. Geologic information that should be considered includes groundwater and seepage conditions; lithology, stratigraphy, and geologic details disclosed by borings, "as-built" records, and geologic interpretation; maximum past overburden at site as deduced from geologic evidence; bedding, folding and faulting; joints and joint systems; weathering; slickensides, and field evidence relating to slides, faults, movements and earthquake activity. Foundations may present problems where they contain adversely oriented joints, slickensides or fissured material, faults, seams of soft materials, or weak layers. Such defects and excess pore water pressures may contribute to instability. Special tests may be necessary to determine physical properties of particular materials. The results of stability analyses afford a means of evaluating the structure's existing resistance to failure and also the effects of any proposed modifications. Results of stability analyses should be reviewed for compatibility with performance experience when possible.

4.4.2.1. <u>Seismic Stability</u>. The inertial forces for use in the conventional equivalent static force method of analysis should be obtained by multiplying the weight by the seismic coefficient and should be applied as a horizontal force at the center of gravity of the section or element. The seismic coefficients suggested for use with such analyses are listed in Figures 1 through 4. Seismic stability investigations for all high hazard category dams located in Seismic Zone 4 and high hazard dams of the hydraulic fill type in Zone 3 should include suitable dynamic procedures and analyses. Dynamic analyses for other dams and higher seismic coefficients are appropriate if in the judgment of the investigating engineer they are warranted because of proximity to active faults or other reasons. Seismic stability investigations should utilize "state-of-the-art" procedures involving seismological and geological studies to establish earthquake parameters for use in dynamic stability analyses and, where appropriate, the dynamic testing of materials. Stability analyses may be based upon either time-history or response spectra techniques. The results of dynamic analyses should be assessed on the basis of whether or not the dam would have sufficient residual integrity to retain the reservoir during and after the greatest or most adverse earthquake which might occur near the project location.

4.4.2.2. <u>Clay Shale Foundation</u>. Clay shale is a highly overconsolidated sedimentary rock comprised predominantly of clay minerals, with little or no cementation. Foundations of clay shales require special measures in stability investigations. Clay shales, particularly those containing montmorillonite, may be highly susceptible to expansion and consequent loss of strength upon unloading. The shear strength and the resistance to deformation of clay shales may be quite low and high pore water pressures may develop under increase in load. The presence of slickensides in clay shales is usually an indication of low shear stength. Prediction

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of field behavior of clay shales should not be based solely on results of
conventional laboratory tests since they may be misleading. The use of
peak shear strengths for clay shales in stability analyses may be uncon-
servative because of nonuniform stress distribution and possible progres-
sive failures. Thus the available shear resistance may be less than if
the peak shear strength were mobilized simultaneously along the entire
failure surface. In such cases, either greater safety factors or residual
shear strength should be used.

4.4.3. Embankment Dams.

4.4.3.1. Liquefaction. The phenomenon of liquefaction of loose,
saturated sands and silts may occur when such materials are subjected
to shear deformation or earthquake shocks. The possibility of lique-
faction must presently be evaluated on the basis of empirical knowledge
supplemented by special laboratory tests and engineering judgment. The
possibility of liquefaction in sands diminishes as the relative density
increases above approximately 70 percent. Hydraulic fill dams in
Seismic Zones 3 and 4 should receive particular attention since such
dams are susceptible to liquefaction under earthquake shocks.

4.4.3.2. Shear Failure. Shear failure is one in which a portion of an
embankment or of an embankment and foundation moves by sliding or rotating
relative to the remainder of the mass. It is conventionally represented
as occurring along a surface and is so assumed in stability analyses,
although shearing may occur in a zone of substantial thickness. The
circular arc or the sliding wedge method of analyzing stability, as per-
tinent, should be used. The circular arc method is generally applicable
to essentially homogeneous embankments and to soil foundations consisting
of thick deposits of fine-grained soil containing no layers significantly
weaker than other strata in the foundation. The wedge method is generally
applicable to rockfill dams and to earth dams on foundations containing
weak layers. Other methods of analysis such as those employing complex
shear surfaces may be appropriate depending on the soil and rock in the
dam and foundation. Such methods should be in reputable usage in the
engineering profession.

4.4.3.3. Loading Conditions. The loading conditions for which the embank-
ment structures should be investigated are (I) Sudden drawdown from spill-
way crest elevation or top of gates, (II) Partial pool, (III) Steady
state seepage from spillway crest elevation or top of gate elevation,
and (IV) Earthquake. Cases I and II apply to upstream slopes only;
Case III applies to downstream slopes; and Case IV applies to both up-
stream and downstream slopes. A summary of suggested strengths and
safety factors are shown in Table 4.

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TABLE 4

FACTORS OF SAFETY /

Case	Loading Condition	Factor of Safety	Shear // Strength	Remarks
I	Sudden drawdown from spillway crest or top of gates to minimum drawdown elevation.	1.2*	Minimum composite of R and S shear strengths See Figure 5.	Within the drawdown zone submerged unit weights of materials are used for computing forces resisting sliding and saturated unit weights are used for computing forces contributing to sliding.
II	Partial pool with assumed horizontal steady seepage saturation.	1.5	$\frac{R+S}{2}$ for $R < S$ S for $R > S$	Composite intermediate envelope of R and S shear strengths. See Figure 6.
III	Steady seepage from spillway crest or top of gates with $K_h/K_v = 9$ assumed**	1.5	Same as Case II	
IV	Earthquake (Cases II and III with seismic loading)	1.0	***	See Figures 1 through 4 for Seismic Coefficients.

/   Not applicable to embankments on clay shale foundation. Experience has indicated special problems in determination of design shear strengths for clay shale foundations and acceptable safety factors should be compatible with the confidence level in shear strength assumptions.

//   Other strength assumptions may be used if in common usage in the engineering profession.

*   The safety factor should not be less than 1.5 when drawdown rate and pore water pressure developed from flow nets are used in stability analyses.

** K_h/K_v is the ratio of horizontal to vertical permeability. A minimum of 9 is suggested for use in compacted embankments and alluvial sediments.

*** Use shear strength for case analyzed without earthquake. It is not necessary to analyze sudden drawdown for earthquake loading. Shear strength tests are classified according to the controlled drainage conditions maintained during the test. R tests are those in which specimen drainage is allowed during consolidation (or swelling) under initial stress conditions, but specimen drainage is not allowed during application of shearing stresses. S tests allow full drainage during initial stress application and shearing is at a slow rate so that complete specimen drainage is permitted during the complete test.

4.4.3.4. <u>Safety Factors</u>. Safety factors for embankment dam stability studies should be based on the ratio of available shear strength to developed shear strength, S_D:

$$S_D = \frac{C}{F.S.} + \sigma \frac{\tan \phi}{F.S.} \tag{1}$$

C = cohesion

ϕ = angle of internal friction

σ = normal stress

The factors of safety listed in Table 4 are recommended as minimum acceptable. Final accepted factors of safety should depend upon the degree of confidence the investigating engineer has in the engineering data available to him. The consequences of a failure with respect to human life and property damage are important considerations in establishing factors of safety for specific investigations.

4.4.3.5. <u>Seepage Failure</u>. A critical uncontrolled underseepage or through seepage condition that develops during a rising pool can quickly reduce a structure which was stable under previous conditions, to a total structural failure. The visually confirmed seepage conditions to be avoided are (1) the exit of the phreatic surface on the downstream slope of the dam and (2) development of hydrostatic heads sufficient to create in the area downstream of the dam sand boils that erode materials by the phenomenon known as "piping" and (3) localized concentrations of seepage along conduits or through pervious zones. The dams most susceptible to seepage problems are those built of or on pervious materials of uniform fine particle size, with no provisions for an internal drainage zone and/or no underseepage controls.

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4.4.3.6. _Seepage Analyses._ Review and modifications to original
seepage design analyses should consider conditions observed in the
field inspection and piezometer instrumentation. A seepage analysis
should consider the permeability ratios resulting from natural deposi-
tion and from compaction placement of materials with appropriate
variation between horizontal and vertical permeability. An under-
seepage analysis of the embankment should provide a critical gradient
factor of safety for the maximum head condition of not less than 1.5
in the area downstream of the embankment.

$$F.S = i_c/i = \frac{H_c/D_b}{H/D_b} = D_b \frac{(\gamma_m - \gamma_w)}{H \gamma_w} \qquad (2)$$

i_c = Critical gradient

i = Design gradient

H = Uplift head at downstream toe of dam measured above
tailwater

H_c = The critical uplift

D_b = The thickness of the top impervious blanket at the
downstream toe of the dam

γ_m = The estimated saturated unit weight of the material in the
top impervious blanket

γ_w = The unit weight of water

Where a factor of safety less than 1.5 is obtained the provision of an
underseepage control system is indicated. The factor of safety of 1.5
is a recommended minimum and may be adjusted by the responsible engineer
based on the competence of the engineering data.

4.4.4. _Concrete Dams and Appurtenant Structures._

4.4.4.1. _Requirements for Stability._ Concrete dams and structures
appurtenant to embankment dams should be capable of resisting over-
turning, sliding and overstressing with adequate factors of safety for
normal and maximum loading conditions.

4.4.4.2. Loads. Loadings to be considered in stability analyses include the water load on the upstream face of the dam; the weight of the structure, internal hydrostatic pressures (uplift) within the body of the dam, at the base of the dam and within the foundation; earth and silt loads; ice pressure, seismic and thermal loads, and other loads as applicable. Where tailwater or backwater exists on the downstream side of the structure it should be considered, and assumed uplift pressures should be compatible with drainage provisions and uplift measurements if available. Where applicable, ice pressure should be applied to the contact surface of the structure at normal pool elevation. A unit pressure of not more than 5,000 pounds per square foot should be used. Normally, ice thickness should not be assumed greater than two feet. Earthquake forces should consist of the inertial forces due to the horizontal acceleration of the dam itself and hydrodynamic forces resulting from the reaction of the reservoir water against the structure. Dynamic water pressures for use in conventional methods of analysis may be computed by means of the "Westergaard Formula" using the parabolic approximation (H.M. Westergaard, "Water Pressures on Dams During Earthquakes," Trans., ASCE, Vol 98, 1933, pages 418-433), or similar method.

4.4.4.3. Stresses. The analysis of concrete stresses should be based on in situ properties of the concrete and foundation. Computed maximum compressive stresses for normal operating conditions in the order of 1/3 or less of in situ strengths should be satisfactory. Tensile stresses in unreinforced concrete should be acceptable only in locations where cracks will not adversely affect the overall performance and stability of the structure. Foundation stresses should be such as to provide adequate safety against failure of the foundation material under all loading conditions.

4.4.4.4. Overturning. A gravity structure should be capable of resisting all overturning forces. It can be considered safe against overturning if the resultant of all combinations of horizontal and vertical forces, excluding earthquake forces, acting above any horizontal plane through the structure or at its base is located within the middle third of the section. When earthquake is included the resultant should fall within the limits of the plane or base, and foundation pressures must be acceptable. When these requirements for location of the resultant are not satisfied the investigating engineer should assess the importance to stability of the deviations.

4.4.4.5. Sliding. Sliding of concrete gravity structures and of abutment and foundation rock masses for all types of concrete dams should be evaluate by the shear-friction resistance concept. The available sliding resistance is compared with the driving force which tends to induce sliding to arrive at a sliding stability safety factor. The investigation should be made along all potential sliding paths. The critical path is that plane or combination of planes which offers the least resistance.

4.4.4.5.1. _Sliding Resistance_. Sliding resistance is a function of the unit shearing strength at no normal load (cohesion) and the angle of friction on a potential failure surface. It is determined by computing the maximum horizontal driving force which could be resisted along the sliding path under investigation. The following general formula is obtained from the principles of statics and may be derived by resolving forces parallel and perpendicular to the sliding plane:

$$R_R = V \tan(\phi + \alpha) + \frac{cA}{\cos\alpha(1 - \tan\phi\tan\alpha)} \tag{3}$$

where

R_R = Sliding Resistance (maximum horizontal driving force which can be resisted by the critical path)

ϕ = Angle of internal friction of foundation material or, where applicable, angle of sliding friction

V = Summation of vertical forces (including uplift)

c = Unit shearing strength at zero normal loading along potential failure plane

A = Area of potential failure plane developing unit shear strength "c"

α = Angle between inclined plane and horizontal (positive for uphill sliding)

For sliding downhill the angle α is negative and Equation (1) becomes:

$$R_R = V \tan(\phi - \alpha) + \frac{cA}{\cos\alpha(1 + \tan\phi\tan\alpha)} \tag{4}$$

When the plane of investigation is horizontal, and the angle α is zero and Equation (1) reduced to the following:

$$R_R = V \tan\phi + cA \tag{5}$$

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4.4.4.5.2. Downstream Resistance. When the base of a concrete structure is embedded in rock or the potential failure plane lies below the base, the passive resistance of the downstream layer of rock may sometimes be utilized for sliding resistance. Rock that may be subjected to high velocity water scouring should not be used. The magnitude of the downstream resistance is the lesser of (a) the shearing resistance along the continuation of the potential sliding plane until it daylights or (b) the resistance available from the downstream rock wedge along an inclined plane. The theoretical resistance offered by the passive wedge can be computed by a formula equivalent to formula (3):

$$P_p = W \tan(\phi + \alpha) + \frac{cA}{\cos\alpha\,(1 - \tan\phi\,\tan\alpha)} \tag{6}$$

P_p = passive resistance of rock wedge

W = weight (buoyant weight if applicable) of downstream rock wedge above inclined plane of resistance, plus any superimposed loads

ϕ = angle of internal friction or, if applicable, angle of sliding friction

α = angle between inclined failure plane and horizontal

c = unit shearing strength at zero normal load along failure plane

A = area of inclined plane of resistance

When considering cross-bed shear through a relatively shallow, competent rock strut, without adverse jointing or faulting, W and α may be taken at zero and 45°, respectively, and an estimate of passive wedge resistance per unit width obtained by the following equation:

$$P_p = 2\,cD \tag{7}$$

where

D = Thickness of the rock strut

4.4.4.5.3. Safety Factor. The shear-friction safety factor is obtained by dividing the resistance R_R by H, the summation of horizontal service

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loads to be applied to the structure:

$$S_{s-f} = \frac{R_R}{H} \tag{8}$$

When the downstream passive wedge contributes to the sliding resistance, the shear fruction safety factor formula becomes:

$$S_{s-f} = \frac{R_R + P_p}{H} \tag{9}$$

The above direct superimposition of passive wedge resistance is valid only if shearing rigidities of the foundation components are similar. Also, the compressive strength and buckling resistance of the downstream rock layer must be sufficient to develop the wedge resistance. For example, a foundation with closely spaced, near horizontal, relatively weak seams might not contain sufficient buckling strength to develop the magnitude of wedge resistance computed from the cross-bed shear strength. In this case wedge resistance should not be assumed without resorting to special treatment (such as installing foundation anchors). Computed sliding safety factors approximating 3 or more for all loading conditions without earthquake, and 1.5 including earthquake, should indicate satisfactory stability, depending upon the reliability of the strength parameters used in the analyses. In some cases when the results of comprehensive foundation studies are available, smaller safety factors may be acceptable. The selection of shear strength parameters should be fully substantiated. The bases for any assumptions; the results of applicable testing, studies and investigations; and all pre-existing, pertinent data should be reported and evaluated.

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